ADDIS ABABA INSTITUTE OF TECHNOLOGY SCHOOL

OF CIVIL AND ENVIRONMENTAL ENGINEERING

CAPACITY EVALUATION OF GOTERA INTERCHANG

By

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A thesis Submitted to school of graduated studies of Addis Ababa University in partial fulfillment of the requirements for the degree of Master of Science in civil engineering.

(Road and Transport Engineering Program)

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Abstract

This thesis addresses the evaluation of operational performance of Gotera interchange in Addis Ababa and the relation between interchange performance measure and capacity in terms of level of service. The capacity and LOS of the interchange is analyzed and evaluated using three performance characteristics (Flow rate in pc/h/ln, Average passenger car speed and Density).

The necessary geometric data for the analysis were obtained from Addis Ababa City Road Authority and site visit where as the available traffic movement data and free flow speed were collected on site. During traffic data collection 25 count stations were selected at which data was recorded for three hours time duration that had been carried out manually during at peak hours based on their vehicle class category, from which vehicles classified into heavy and light vehicles in order to compute percentage of heavy vehicles which is the main factors that affects hourly flow rate.

Based on Capacity and Level of service analysis results, the current condition of interchange from 25 lines 12 lines have level of service of A and 7 lines have level of service of B, the rest 6 lines have LOS C and the line which carries maximum traffic flow is the main lines of the interchange flows Debre-Zeyt to Meskel- Square which is designated by D-M(P5-p8) carries 1,027pc/hr/ln and line which carries the least traffic flow is Bole to Debre-Zeyt designated by B-D(P10) which carries only 100pc/hr/ln.

Results of merge influence of area also summarized as three merge areas have high flow rate from seven points that are towards Kera designated by exit of P12′ and P13′ and towards Meskel square designated by Exit of p3″and p1′ and Exit of p11′ have 2523, 2540 and 2426 pc/hr/ln respectively and all the three points have LOS of C.

In the future after 10 years, LOS of the interchange lines will be between within A-F grade. One line will have A, four lines will have B, four lines will have C, five lines will have D, six lines will have E and five lines will have F. As shown as results of merge area, two merge influence area will have LOS of C, three will have D and two will have E, these influence area will have LOS E are exit of Meskel Square and Kera .Currently, the overall condition of the interchange is safe and to the future after 10 years, will not give a satisfactory service.

1. Introduction

Freeways play one of the most important roles in the current transportation system because they can carry a very large portion of traffic and provide high speed operation. Freeways provide important access to the public for their daily lives and routes for trucks for commercial activities. Interchanges are important parts of the freeway system and it is a system of interconnecting roadways in conjunction with one or more grade separations that provides for the movement of traffic between two or more roadways or highways on different levels.

Interchanges achieve three objectives in the freeway system. First, interchanges provide a method to finish the traffic transformation between local streets and freeways or two freeways that at-grade intersections cannot achieve. Second, interchanges are good ways to reduce conflict points and improve the safety of the traffic movements, and they also can add extra capacity to the system. Third, interchanges improve the efficiency of traffic operations and reduce the delays at intersections

Evaluation of freeway capacity is very important since it is directly related to delay, level of service, accident, operation cost, and environmental issues. This thesis contains a methodology for evaluating of the capacity and level of service (LOS) of Gotera interchange by analyzing geometric and traffic data. The analysis must consider a wide variety of prevailing conditions, including the amount and distribution of traffic movements, traffic composition, geometric characteristics and social behavior. The methodology focuses on the determination of LOS, capacity and other performance measures for lane groups and intersection approaches as a whole before and after the implementation of the project and also to the future.

1.1. Statement of the problem

Traffic congestion at exit of Meskel- Square and towards Kera is common during peak hour at Gotera interchange. This can be due to driver behavior, inadequate road (junction planning) or geometric conditions traffic. This problem will continue and it may worsen in the future due to the rapid growth of population and vehicle numbers in Addis Ababa. Therefore, it is essential to evaluate LOS and capacity of the interchange for proper traffic operation.

1.2. Objectives of the Study

General objectives of the Study

- To evaluate the present capacity of the interchange by capacity analysis and LOS
- To determine the capacity of the interchange for future (after ten years) traffic flow.
- To draw conclusions and recommendations for possible future considerations of the interchange

Specific objective

- Determine LOS and capacity of the exit of Meskel- Square and towards Kera
- To evaluate the present capacity of the interchange by capacity analysis and LOS
- To determine the capacity of the interchange for future traffic flow.
- To draw conclusions and recommendations for possible future considerations of the interchange

1.3. Organization of the thesis

The thesis has been divided in to six chapters. The introductory chapter includes, introduction, Statement of the problem, objective of the thesis. The second chapter gives a brief literature review which discuses about general characteristics of interchange, freeway highway characteristics of basic freeway and ramp section and their performance evaluation methods, criteria which used to evaluate their capacity.

Study area description, data collection method and data analysis are deal with in chapter three. In the fourth chapter the analysis results and discussions are explained. Chapter five includes traffic forecasting of the interchange and determines the capacity of the interchange after ten years. Conclusion and recommendation parts are drown in chapter six.

2. Literature review

2.1. Grade separation and Interchange

General

Intersections can be at grade, grade separation and interchange. Intersection at grade can be eliminated by the use of grade –separation structures that permit the cross flow of traffic at different levels without interruption and increase in safety for traffic movement to eliminate bottlenecks. There are several basic interchange configurations to accommodate turning movements at a grade separation. The type of configuration is determined by the number of intersection legs, expected volumes of through and turning movement's type of truck traffic, topography, culture, design control and the number of legs at the interchange. These interchanges are most appropriate at locations where the intersecting facility is classified as a local, collector, freeway or arterial. (AASHTO, 2004)

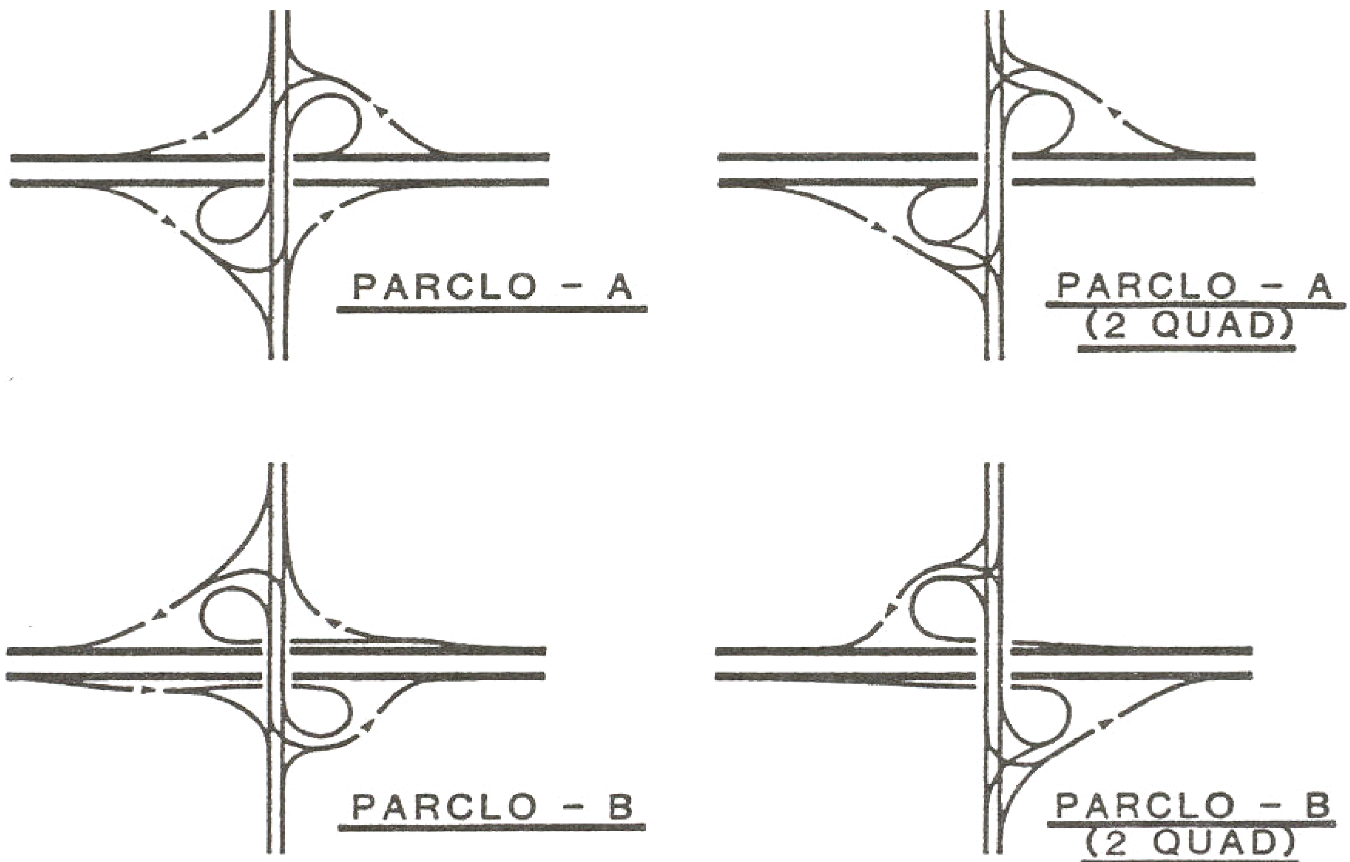

Figure 2.1: Parcol interchange Types (Partial cloverleaf) (AASHTO, 2004)

2.2. Freeway Interchange Characteristics

Freeways interchange plays one of the most important roles in the current transportation system because they can carry a very large portion of traffic and provide high speed operation. Freeways provide important access to the public for their daily lives and routes for trucks for commercial activities. A free way is a divided highway with full access control and two or more lanes in each direction for the exclusive use of moving traffic. A free way is composed of three elements basic free way sections, weaving area and ramp junctions. These sections are segments of the free way that are outside of the influence area of ramps or weaving areas. Merging and diverging occurs where on - or off- ramps join the basic free way section. The exact point at which a basic freeway section begins or ends-that is where the influence of weaving areas and ramp junctions has dissipated – depends on local conditions, particularly the level of service operating at the time (HCM, 2000)

Interchanges are important parts of the freeway system and it is a system of interconnecting roadways in conjunction with one or more grade separations that provides for the movement of traffic between two or more roadways or highways on different levels. Interchanges achieve three objectives in the freeway system. First, interchanges provide a method to finish the traffic transformation between local streets and freeways or two freeways that at-grade intersections cannot achieve. Second, interchanges are good ways to reduce conflict points and improve the safety of the traffic movements, and they also can add extra capacity to the system. Third, interchanges improve the efficiency of traffic operations and reduce the delays at intersections. (Hantao Zhong,2012)

Now a day, interchanges have become the critical points in many designs. They determine the delay, speed, and capacity; have higher collision rates; and cost more than roadway segments. They are influenced by lots of aspects. Designers always need to take into account the right-of-way (ROW), increasing average daily traffic or peak hour traffic, driver expectations, wrong way potential, and the surrounding environment especially in developing cities, the available land becomes more and more precious and expensive (Hantao Zhong,2012).

2.2.1. Basic Freeway Sections

General Basic Freeway Sections Performance Evaluation Concepts

A conceptual approach for analyzing and evaluating free flow interchanges has the following components (HCM, 2000)

1. Capacity and flow ratio
2. Level of service (LOS).

   Freeway is the highest form of dual carriage way roads with two or more lanes for the exclusive use of traffic in each direction and full control of access and egress. full control of access and egress means that through traffic on the main line has priority of movement , at grade crossing and private driveway connections are prohibited and movements onto or off the main line are only possible by way of specific roads or ramp

2.2.1.1. Freeway Interchange Demand and Capacity

Demand is the principal measure of the amount of traffic using a given facility and it relates to vehicles arriving, while volume relates to vehicles discharging. Traffic demand varies by month of the year, day of the week, hour of the day, and sub hourly interval within the hour. These variations are important if highways are to effectively serve peak demands without breakdown. The effects of a breakdown may extend far beyond the time during which demand exceeds capacity and may take up to several hours to dissipate. Thus, highways minimally adequate to handle a peak-hour demand may be subject to breakdown if flow rates within the peak hour exceed capacity (HCM, 2000).

The capacity of freeway is the maximum sustained 15-min rate of flow, expressed in passenger cars per hour per lane (pc/h/ln), which can be accommodated by a uniform freeway segment under prevailing traffic and road way condition in one direction. The road way condition Include number and width of lanes right shoulder lateral clearance; interchange spacing and grade .The traffic conditions are flow characteristics including the percentage compositions of vehicle types and the extent to which drivers are familiar with the freeway segment. Conditions of free flow speed occur when flow rates are low to moderate (less than 1300 pc/h/ln at 120km/h). As flow rates increase beyond 1300, the mean speed of passenger cars in the traffic stream decreases. The basic characteristics of uninterrupted flow were presented Capacity analysis procures for freeways and multilane highways are based on calibrated speed - flow curves for sections with various free-flow speeds operating under basic conditions (Roess,3rd edition).

Peak Hour and Analysis Hour

Capacity and other traffic analyses focus on the peak hour of traffic volume, because it represents the most critical period for operations and has the highest capacity requirements. The peak-hour volume, however, is not a constant value from day to day or from season to season. If the highest hourly volumes for a given location were listed in descending order, a analysis hour large variation in the data would be observed, depending on the type of facility.

The selection of an appropriate hour for planning, design, and operational purposes is a compromise between providing an adequate level of service (LOS).

As a general guide, the most repetitive peak volumes may be used for the design of new or upgraded facilities. The LOS during higher-volume periods should then be tested as to the acceptability of the resulting traffic conditions. A peak hour or rush hour is a part of the day during which traffic congestion on roads and crowding on public transport is at its highest. Normally, this happens twice every weekday-once in the morning and once in the evening, the times during when the most people commute. The term is often used for a period of peak congestion that may last for more than one hour.

The relationship between the peak 15-min flow rate and the full hourly volume is given by the peak-hour factor (PHF). Whether the design hour is measured, established from the analysis of peaking patterns, or based on modeled demand, the PHF is applied to determine design-hour flow rates. PHFs in urban areas generally range between 0.80 and 0.98. Lower values signify greater variability of flow within the subject hour, and higher values signify less flow variation. PHFs over 0.95 are often indicative of high traffic volumes, sometimes with capacity constraints on flow during the peak hour.

Calculation of Analysis Period Flow: - These are based on peak 15-min flow rates. Because traffic does not flow evenly over an hour, sub hourly peaking should be accounted for when the analysis is in terms other than 15-min flows. The relationship between the peak 15-min flow rate and the full hourly volume is given by the peak-hour factor (PHF). To convert peak 15-min flow rates to hourly volumes, the flow rate is multiplied by the PHF. Most of the analytical procedures use the peak 15-min flow rate. This rate is obtained by dividing the hourly volume by the PHF. Service volume results, expressed in 15-min flow rates, must be multiplied by the PHF to obtain the equivalent hourly volume (Roess,3rd edition)

Composition of Traffic and Vehicle Equivalents: - Vehicles of different sizes and weights have different operating characteristics that should be considered in highway design .Besides being heavier; trucks are generally slower and occupy more road way space. Consequently, trucks have a greater individual effect on highway traffic operation than do passenger vehicles. The presence of vehicles other than passenger cars such as trucks, buses, and recreational vehicles in a traffic stream reduces the maximum flow on the high way because of their size, operating characteristics, and interaction with other vehicles, because freeway capacity is measured interims of pc/h/ln the number of heavy vehicles in the traffic stream must be converted into an equivalent number of passenger cars .The effect on traffic operation of one truck is often equivalent to several passenger cars. Traffic volumes containing a mix of vehicle types must be converted into an equivalent flow of passenger cars using passenger car equivalents (PCEs). Thus, the larger the proportion of trucks in a traffic stream, the greater the equivalent traffic demand and the greater the highway capacity needed. (Anthony Ingle, 2004).

Analysis Methodologies for Basic Freeway Sections

Maximum densities, minimum average speeds, maximum v/c ratios, and maximum service flow rates for the various levels of service for freeways and multilane highways. Analysis methodologies are provided that account for the impact of variety of prevailing conditions including :Lane width, Lateral clearance, Number of lanes, Type of median, Frequency of interchange and Presence of heavy vehicles in the traffic stream(Roess,3rd edition).

Operational Analysis:- This is the common form of analysis in which all traffic , road way and control conditions are defined for an existing or projected highway section and the expected level of service and operating parameters are determined and convert the existing and forecast demand volumes to an equivalent free flow rate under ideal conditions .

$$\mathrm{Vp} = \mathrm{V}/(\mathrm{PHF} * \mathrm{N} * \mathrm{fHV} * \mathrm{fp}) \qquad (2\text{-}1)$$

Where

VP = demand flow rate under equivalent ideal conditions, pc/h/ln

PHF = Peak –hour factor

N=Number of lanes (in one direction) on the facility

fHP = Heavy vehicle adjustment factor

fp = Adjustment factor for presence of occasional or non-familiar users of a facility

2.2.1.2. Level of Service (LOS) For Freeway Sections

Level of service (LOS) qualitatively measures both the operating conditions within a traffic system and how these conditions are perceived by drivers and passengers. The analysis must consider a wide variety of prevailing conditions, including the amount and distribution of traffic movements, traffic composition, geometric characteristics, and details of intersection. The methodology addresses the capacity, LOS, and other performance measures for lane groups and intersection approaches and the LOS for the intersection as a whole. Capacity is evaluated in terms of the ratio of demand flow rate to capacity (v/c ratio), whereas the measure of effectiveness used to define level of service is density. The use of density rather than speed is based primarily on the shape of the speed – flow relationships. (HCM, 2000)

LOS is determined by using different approaches as recommended by the HCM, namely for two-lane and multi-lane freeways. LOS is indicated by using the letters of the alphabet (A through to F) A representing free flow that the best operating conditions, B represents reasonably free flow , C represent stable flow but most drivers are restricted in their freedom to select their own speed. (AACRA, 2003), D represent approaching stable flow, E represents Volatile flow and F represents unstable flow. When new road infrastructure is designed, most public sector entities tend to require a design LOS of at least 'C' in the design year – in other words, if a facility is designed to last for a period of seven years, the facility should preferably still operate at a LOS of 'C'. (Africon Ltd 2011).

The three measures of speed, density and flow or volume are interrelated.. Level of service for a basic freeway segments are summarized below.

Table 2.1: Level of service Criteria for basic freeways section (HCM, 2000)

LOS	Density Range for Basic Freeway Sections(pc/km/ln
A	0-7
B	>7-11
C	>11-16
D	>16-22
E	>22-28
F	>28

The measure of effectiveness used to define level of service is density. The use of density, rather than speed, is based primarily on the shape of the speed –flow relationships. Because average –speed remains constant through most of the range of flows and the total difference between free-flow speed and the speed at capacity is relatively small. The following figure shows speed-flow relation- ships at basic freeway section. (Roess, 3rd edition)

According to the HCM 2000, Free-Flow Speed (FFS) is the mean speed of passenger cars measured during low to moderate flows in the field.

The actual level of service to use will depend on a range of factors including the overall strategy for the road in question available funds and the relationship of the road reminder of the system. Level of service 'C' for a design year 20-25 years away may be a reasonable target. (AACRA, 2003)

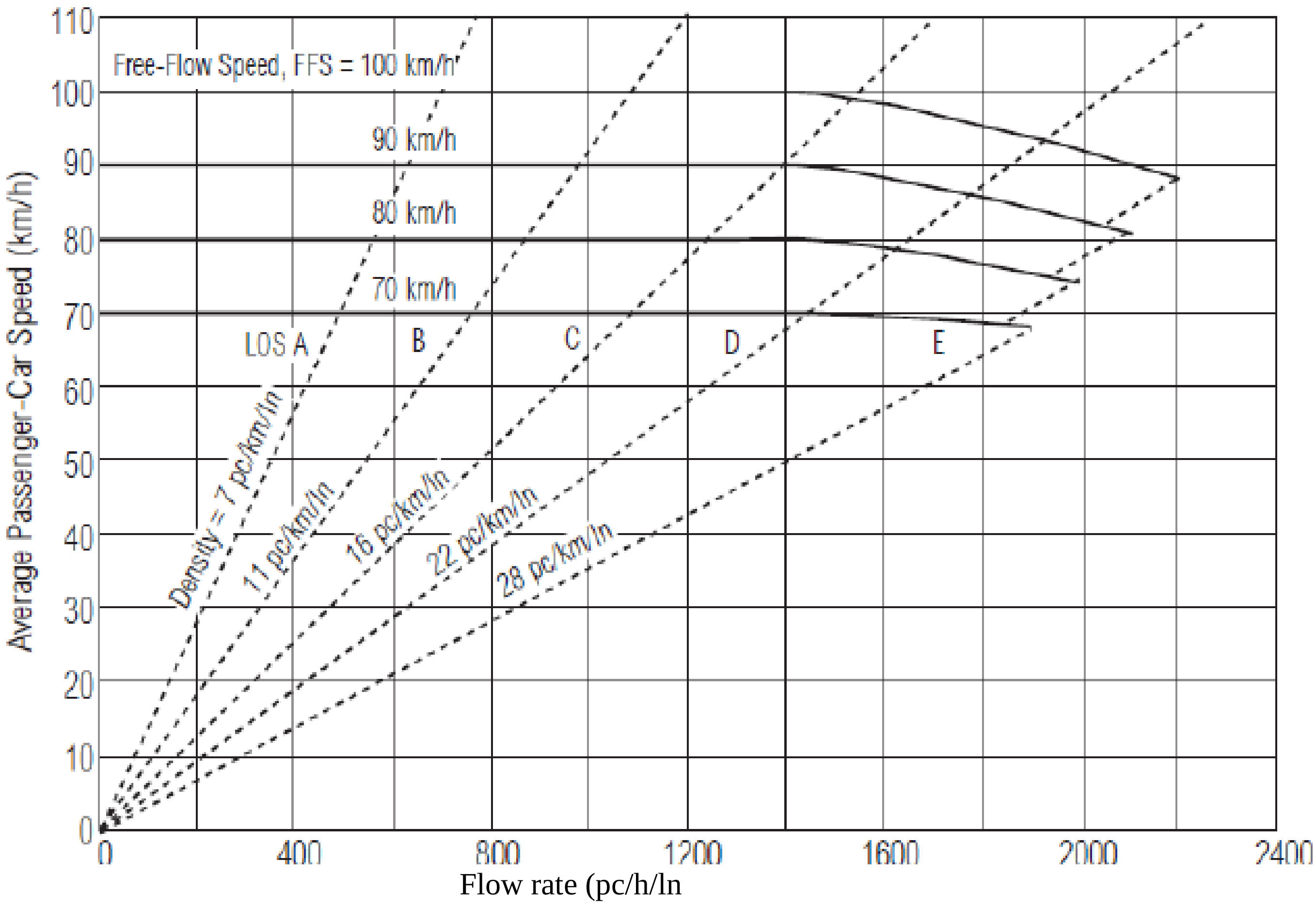

Figure 2.2: Speed- flow curves and level of service for basic freeway segment (HCM, 2000)

Although speed- flow -density relationship are the principal factor affecting the level-of- service of a high way segment under ideal condition such as lane width, lateral obstruction, traffic composition, grade, speed, and driver population. Evaluate Level of service of a basic freeway sections, the following three performance characteristics can be described

$V_{P:}$ Flow rate in passenger, pc/h/ln

S: Average passenger car speed, km/h

D: Density (number of cars per km), pc/km/ln

The relationship between the three performance characteristic is

$$D = \text{Vp/S} \tag{2-2}$$

For $90 \leq FFS \leq 120$ and flow rate (Vp)

$(3100 - 15FFS) < Vp \leq (1800 + 5FFS)$

$$S = FFS - [1/28 * (23FFS - 1800) \frac{(Vp + 15FFS - 3100)]\text{^}2.6}{20FFS - 1300} \quad \text{(2-3}$$

For $90 \leq FFS \leq 120$ and

$VP <$ (3100-15FFS), $S = FFS$

The average of all passenger-car speeds measured in the field under low- to moderate- volume conditions can be used directly as the FFS of the freeway segment.

Determining the free- flow speed

The free –flow speed of a facility is best determined by field measurement during low to moderate flow rate, but it is not always possible to measure it. The average of all passenger-car speeds measured in the field under low- to moderate- volume conditions can be used directly as the FFS of the freeway segment

Freeways the free-flow speed of a freeway is estimated as

$$FFS = BFFS - fLW - fLC - fN - fID \tag{2.4}$$

Where FFS = free-flow speed of the freeway, km/h

$BFFS$ = base free-flow speed of the freeway (110km/h for urban and suburban freeways, 120km/h for rural freeways.

Lane width adjustment

Table 2.2: Adjustment to Free-flow speed for lane width on freeway

Lane width(m)	**Lane adjustment (fLW)**
3.6	0.0
3.5	1.0
3.4	2.1
3.3	3.1
3.2	5.6
3.1	8.1
3.0	10.6

Lateral Clearance Adjustment:

Base lateral clearance is 1.8m or greater on the right side and 0.6m or grater on the median, or left side of the freeway section. There is no adjustment provided for median clearance less than 0.6m.

Table 2.3: Adjustment to free-flow speed for Lateral Clearance on freeway

Right shoulder lateral clearance	Adjustment of lateral clearance			
	Lane in one direction			
	2	3	4	>=5
>=1.8	0	0	0	0
1.5	1.0	0.7	0.3	0.2
1.2	1.9	1.3	0.7	0.4
0.9	2.9	1.9	1.0	0.6
0.6	3.9	2.6	1.3	0.8
0.3	4.8	3.2	1.6	1.1
0.0	5.8	3.9	1.9	1.3

Adjustment for Number of Lanes

The base condition for number of lanes in one direction on a freeway is five or more lanes. Adjustment is not recommended to rural freeways, however, continued application of adjustment to urban and suburban freeways are controversial.

Table 2.4: Adjustment to Free flow speed for number of lanes on a freeway

Number of lanes in one direction	Adjustment factor(km/h)
>=5	0.0
4	2.4
3	4.8
2	7.3

Interchange Density Adjustment

The most significant impact on freeway free-flow speed is the number and spacing of interchanges. The interchange density is not based on the number of ramps and it may consist of several ramp connections it may be diamond interchange which has four ramps, full cloverleaf interchange has eight. To qualify as interchange there must be at least one on- ramp where as a junction with only off-ramp are not qualify as an interchange. The base condition for interchange density is 0.8 interchanges /km which imply an average interchange spacing of two miles.

Table 2.5: Adjustment to free-flow speed for interchange density on a freeway

Interchange per km	Adjustment factor for free flow speed (km/h)
<=0.3	0.0
0.4	1.1
0.5	2.1
0.6	3.9
0.7	5.0
0.8	6.0
0.9	8.1
1.0	9.2
1.1	10.2
1.2	12.1

Determining Flow Rate

The hourly flow rate must reflect the influence of heavy vehicles the temporal variation of traffic flow over an hour and the characteristics of the driver population. These effects are reflected by adjusting hourly volume or estimates reported as vehicles per hour to arrive at an equivalent passenger car flow rate is calculated using the heavy –vehicle and peak hour adjustment factor and is written as pc/h/ln(2) as shown equation(2-1)

$$Vp = V/(PHF * N * fHV * fp)$$

Where:-

Vp = 15-min passenger –car equivalent flow rate (pc/h/ln)

V = hourly volume

PHF = Peak –hour factor

N =Number of lanes (in one direction) on the facility

fHV = Heavy vehicle adjustment factor

fp = Adjustment factor for presence of occasional or non-familiar users of a facility

Peak-hour Factor

PHF is the relationship between the peak 15-minute flow rate and the full hourly volume. Peak-hour factors for freeways range between 0.80 and 0.95. (Africon Ltd, 2011)

The peak hour factor (PHF) represents the variation in traffic flow within an hour. On the freeway, typical PHF ranges from 0.8-0.95 from which low PHF are characteristics of rural highways and higher PHF factors are typically urban and suburban highways. Generally 0.92 is recommended for urban and suburban highways. (HCM, 2000)

The analysis of level of service is based on peak rates of flow occurring within the peak hour because substantial short-term fluctuations typically occur during an hour. Common practice is to use a peak 15-minute rate of flow. Flow rates are usually expressed in vehicles per hour, not vehicles per 15 minutes. The relationship between the peak 15-minute flow rate and the full hourly volume is given by the peak-hour factor (PHF) as shown in the following equation (Roess,3rd edition). If 15-minute periods are used, the PHF is computed as:

$$PHF = V/(4 \times V_{15}) \qquad (2\text{-}5)$$

Where

V =peak-hour volume (vph)

V15 = volume during the peak 15 minutes of flow (veh/15 minutes)

Heavy Vehicle Adjustment Factors

Freeway traffic volumes that include a mix of vehicle types must be adjusted to an equivalent flow rate expressed in passenger –cars per hour per lane.

$$fHV = 1/(1 + PT(ET - 1) + PR(ER - 1)) \qquad (2\text{-}6)$$

Where fHV = Heavy vehicle adjustment factor

PT = Portion of trucks and buses

PR = Portion of recreational vehicles

ET = Passenger car equivalency for trucks

ER = Passenger car equivalency for recreational vehicle

HCM considers trucks and buses to have the same PCE because trucks are generally the only heavy vehicle type present in the traffic stream as shown table (2.6). PCEs are used to convert a mixed vehicle flow into a passenger car only flow with the same operating speed.

Traffic composition

It is not practical to design for a heterogeneous traffic stream and, for this reason, trucks and other types of vehicles are converted to equivalent Passenger Car Units (PCUs). Furthermore, PCU for different vehicle classes in Addis Ababa city were reviewed and recommend as shown below. (ERC, 2015)

Table 2.6: Recommended passenger car equivalent factor

Vehicle type	passenger car equivalent factors		
	Minimum	Maximum	Suggested value
Bicycle	0.2	0.4	0.3
Moto cycles	0.2	0.64	0.4
Cars and vans	1.0	1.0	1.0
Minibus (4 tyres)	1	1.26	1.1
Bus(>4tyers)	1.5	3.6	2.25
Goods(>4tyers)	1.6	2.8	2.1

1965 HCM formally introduced both the Level of Service (LOS) concept and the term Passenger Car Equivalent (PCE). LOS was defined in terms of two parameters: operating speed and volume-to-Capacity ratio. PCE for heavy vehicles was the number of passenger cars displaced in the traffic flow by a truck or a bus, under prevailing roadway and traffic conditions. HCM 2000 also uses a single PCE of 1.5 for level freeways.

Freeway data and methodology

Highway capacity software (HCS2000) is used to evaluate freeway mainline /merge/diverge segments.

In general the following table provide the assumptions for freeway main line, freeway merge/diverge intersections (Eric shimizu /CH2M HILL , 2003)

Table 2.7 Assumptions for freeway mainline and merge/diverge analysis

Freeway merge/diverge		
	Freeway data	
	Number of lanes on freeway	2
	Free flow speed (mph) S_{FF}	Based on adjacent mainline section
	Peak hour factor	Based on data otherwise 0.90
	terrain	Level or rolling grade analysis may be required
	Trucks and buses (%)	Based on data
	Rvs(%)	0
	Driver population adjustment fp	(0.90 to 1.00)recommended 0.98
	On ramp data	
	Free flow speed (mph) S_{FF}	-35-regular ramps -25 loop ramps
	Number of lanes on ramp	1-2
	Length of 1st acc. lane	TBD
	Length of 2nd acc. lane	TBD
	Peak hour factor	Based on data otherwise 0.90
	terrain	Level or rolling grade analysis may be required
	Trucks and buses (%)	Based on data
	Rvs(%)	0
	Driver population adjustment fp	(0.90 to 1.00) recommended 0.98
	Off ramp data	
	The same as on ramp data	

Generally calculating the flow rate for a basic freeway sections describe as equation (2-1).

2.2.2. Ramps

Ramp is one part of freeway interchanges which can be divided into three kinds: direct ramp, indirect ramp, and loop ramp. Direct ramps are widely used to achieve right turn movements in interchanges with high design speeds and low costs. Indirect ramps are often used for left turn movements. However, an indirect ramp requires a high cost and may use a large right-of-way. (Hantao Zhong,2012)

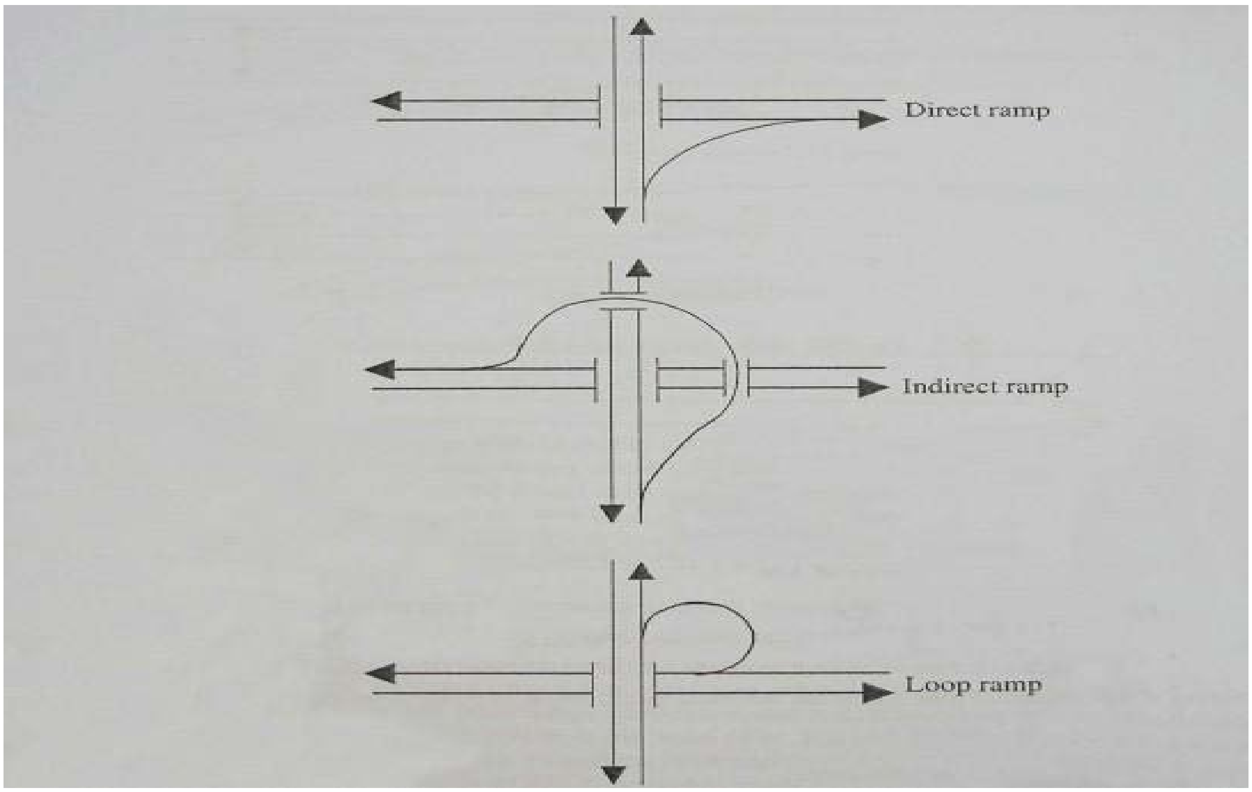

Figure 2.3: Types of Ramps *(*Hantao Zhong, 2012)

Ramp design must account for speed changes and transitions to and from the interchanging facilities. Service interchanges typically require transitioning to a stop condition and providing adequate storage for queued vehicles. System interchange ramp configurations often require special attention to grade separations as well as ramp and freeway levels. Drivers expect high speeds on ramps at a system interchange. Indirect ramps are mostly used in high-volume conditions.

2.2.3. Loop ramp

A loop ramp is a very common type of ramp in a system interchange to achieve left turn movements. Loop ramps need large rights-of-way and it is rare to have a loop ramp with a high design speed. Loop ramp designs require applying speed transition principles for decelerating and accelerating traffic. These three-dimensional considerations can affect horizontal ramp placement to attain desired grades. Interchanges with loops in all four quadrants are referred as full clover leafs and all others are “partial cloverleaf”.

A full cloverleaf may not be warranted at major-minor crossings where, with the provision of only two loops, freedom of movement for traffic on the major road can be maintained by confining the direct at-grade left turns to the minor road. The principal disadvantages of the cloverleaf are the additional travel distance for left-turning traffic, the weaving maneuver generated, the very short weaving length typically available, and the relatively large right-of-way areas needed.

The travel distance on a loop, as compared with that of a direct left turn at grade, increases rapidly with an increase in design speed. On a loop designed for 30 km/h (30-m radius the extra travel distance is approximately 200 m.. Travel time on loops varies almost directly with the design speed, the increased speed being more than balanced by increased distance. For an increase of 10 km/h in loop design Speed, travel time increases 20 to 30 percent. This increase in travel time is actually somewhat less when the overall maneuver is considered because of deceleration and acceleration outside the limits of the loop proper. (In any case, the travel time via a loop may be much less than the travel time when making a direct left turn). (Hantao Zhong,2012).

2.2.4. Partial Cloverleaf Ramp Arrangements

In the design of partial clover leafs, the site conditions may offer a choice of quadrants to use. However, at a particular interchange site, topography and culture may be the factors that determine the quadrants in which the ramps and loops can be developed. Ramps should be arranged so that the entrance and exit turns create the least impediment to the traffic flow on the major highway.

2.2.5. Ramp Junctions and Their Capacity

Ramp junctions may be merge or diverge and their capacity area is always controlled by the capacity of its entering and exiting roadways (the capacity of the ramp itself).The operation of vehicles within the ramp influence area; the following are the major inputs (HCM, 2000).

- Geometric data
- Free-flow speed and
- Demand.

In addition to the above inputs other demand adjustments are required to analyze flow rate of the road section. These are Peak hour factor, heavy vehicle factor and driver population factor. Computation of flow rate is also influenced by the area at which the data is taken may be merge or diverge. Capacity values are determined and compared with existing or forecast demand flows to determine the likelihood of congestion. Several capacity values are evaluated.

- Maximum total flow approaching a merge or diverge area on the freeway (v_F),
- Maximum total flow departing from a merge or diverge area on the freeway (v_{FO}),
- Maximum total flow entering the ramp influence area (v_{R12} for merge areas and V_{12} for diverge areas), and
- Maximum flow on a ramp (v_R).

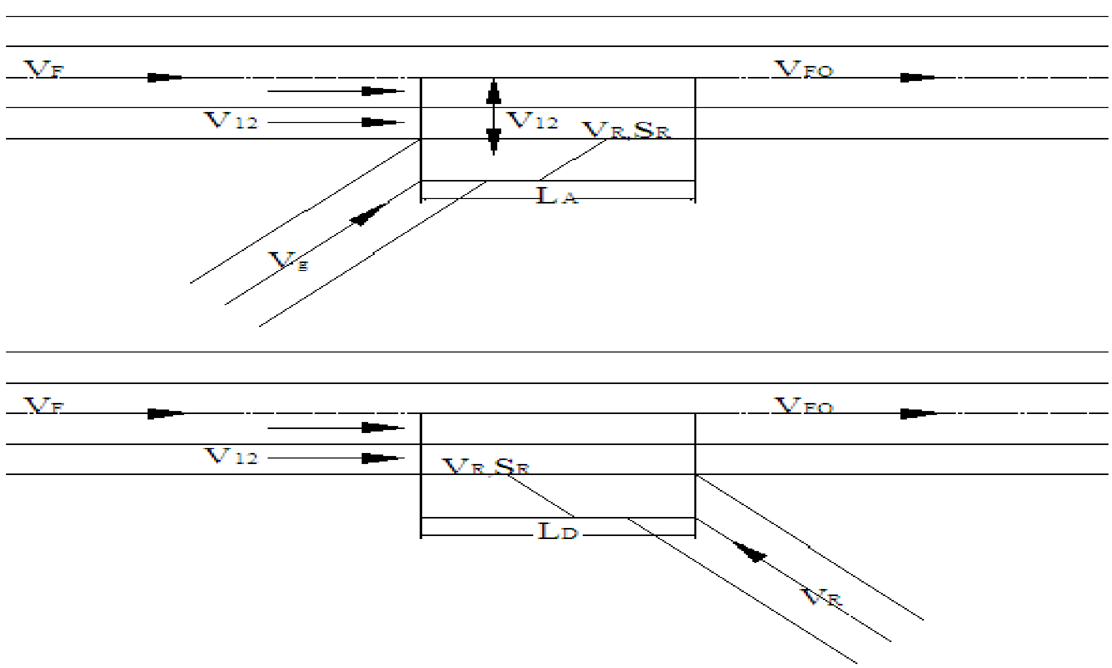

Figure 2.4: Merge and Diverge Influence area (HCM, 2000)

After the above variables are obtained the flow density and level of service of the ramp influence area are computed. A critical geometric parameter influencing operations at merge or diverge area is the length of the acceleration or deceleration lane. This length is measured from the point at which the left edge of ramp lane and the right edge of the freeway lanes converge to the edge of the taper segment connecting the ramp to the freeway. The following figure shows the ramp influence areas and key variables and their relationships to each other.

All aspects of the model and LOS criteria are expressed in terms of equivalent maximum flow rates in passenger cars per hour (pc/h) under base conditions during the peak 15 min of the hour of interest. Therefore, before any of these procedures are applied, all relevant freeway and ramp flows must be converted to equivalent pc/h under base conditions during the peak 15 min of the hour. (HCM, 2000)

$$vi = Vi/(PHF \ * fHV * fp) \qquad (2\text{-}7)$$

Where

vi = flow rate for movement i under base conditions during peak 15 min of= hour (pc/h),

Vi = hourly volume for movement i (veh/h),

Other adjustment factors are the same as those used for analysis of basic freeway segments.

Operational characteristics of ramp road way differ from the freeway main line.

- They are roadways of limited length and width (often just one lane);
- Free-flow speed is frequently lower than that of the roadways connected
- On single-lane ramps, where passing is not possible, the adverse impact of trucks and

  Other slow-moving vehicles is more pronounced and

Two-lane on-ramps can accommodate more than 2,250 to 2,400 pc/h through the merge area itself. The two-lane configuration will achieve a merge with less turbulence and a higher LOS but will not increase the capacity of the merge, which is controlled by the capacity of the downstream freeway segment. For higher on-ramp flows, a two-lane on-ramp must be used in conjunction with a lane addition and a major merge configuration. Two-lane off-ramps can accommodate higher ramp flows through the diverge area than can single-lane off-ramps

Ramp Lane Numbers

If a single lane ramp on a level grade is more than 300m long consideration maybe given to making it two lanes wide once past the exit so that slower vehicles may be over taken. The ramp may merge back to one lane before it joins the through carriageway if it is a free flow ramp, or join as a two lane entry if the free way is to be widened by a lane. Two lane ramps (one lane at the nose) are also used at higher traffic volumes. Once design volumes exceed 1800 PCU/h, two lane ramps with shoulder may be required (AACRA, 2003)

Table 2.8: Ramp lane numbers based on AACRA

Type of ramp	Design Hourly Volume (PCU/hr)
Sigle Lane Ramp <300m Long	<1000
Single Lane Loop	<900
Dual lane loop, lane of at the nose	<1500
Dual lane ramp, one lane at the nose	<1800
Two lane ramp	>1800

Table 2.9: Capacity of the ramp roadway based on HCM

Free-Flow Speed of Ramp, SFR (km/h)	Capacity (pc/h)	
	Single-Lane Ramps	Two-Lane Ramps
>80	2200	4400
>65-80	2100	4100
>50-65	2000	3800
>=30-50	1900	3500
<30	1800	3200

Level of service (LOS)

LOS in merge and diverge influence areas is determined by density for all cases of stable operation

Table 2.10: LOS criteria for merge and diverge area (HCM, 2000)

LOS	Density (pc/km/ln)
A	<=6
B	>6-12
C	>12-17
D	17-22
E	<30
F	Demand exceeds capacity

2.2.5.1. Merge Influence Area

The principal influences on flow on merge influence area are

- Total free flow approaching merge area (vf)
- Total ramp flow (v_R)
- Total length of acceleration(lA)
- Free flow speed of ramp at point of merge area

The following table lists equations used for predicting v_{12} (flow entering lane 1 and 2 as shown figure 2.5) immediately up stream of the ramp influence area .These equations apply to six and eight lane freeways (three and four lane in each direction respectively). For four lane freeways (two lane in each direction) only lane 1 and 2 exists and V_{12} =V_F

Table 2.11: Models for predicting V_{12} at on ramp

	$V12 = VF * PFM$ (2-8)
4-lane freeway(2 lanes -each direction)	$PFM = 1.00$ $(2-9)$
6-lane freeway(3 lanes-each direction)	$P\ FM = 0.5775 + 0.000092LA$ (2-10) $P\ FM = 0.7289 - 0.0000135(vF + vR) - 0.002048SFR + 0.0002Lup$ $(2-11)$ $P\ FM = 0.5487 + \frac{0.0801VD}{Ldown}$ (2-12)
8-lane freeway(4 lanes-each direction)	$P\ FM = 0.2178 - 0.000125VR + 0.05887LA/\ SFR$ (2-13)

V_{12}=flow rate in lane 1 and 2 of freeway immediately upstream of merge (pc/h)

V_F=free way demand flow rate immediately upstream of merge (pc/h)

V_R=on-ramp demand flow rate

V_D=demand flow rate on adjacent downstream ramp

P_{FM}=proportion of approaching freeway flow remaining in lane 1 and 2 immediately upstream of merge

L_A = length of acceleration lane

S_{FR} = free flow speed of ramp

L_{UP} = distance to adjacent upstream ramp

L_{down} = distance to adjacent downstream ramp

Determining Capacity of Merge Area

The capacity of merge area is determined primarily by the capacity of the downstream freeway segment. Thus, the total flow arriving on the upstream freeway and the on-ramp cannot exceed the basic freeway capacity of the departing downstream freeway segment (HCM, 2000). For an on ramp, the flow entering the ramp influence area include

$$VR12 = V12 + VR \tag{2-14}$$

The following table shows capacity flow rates for the total downstream freeway flow ($V = VF + VR$) and maximum desirable values for the total flow entering the ramp influence area$V_{R12.}$

Table 2.12: Capacity values for merge areas

Freeway Free-flow speed(km/h)	Maximum Downstream freeway flow(pc/h)				Max Desirable flow entering influence area ,VR12(pc/h)
	Number of lane in one direction				
	2	3	4	>4	
120	4800	7200	9600	2400/ln	4600
110	4700	7050	9400	2350/ln	4600
100	4600	6900	9200	2300/ln	4600

Determining LOS

LOS criteria for merge areas are based on density in the merge influence area as shown table 2.12 and the following equation is used to estimate density in the merge influence area under saturated flow condition.

$$DR = 3.402 + 0.00456VR + 0.0048V12 - 0.01278LA \qquad (2\text{-}15)$$

Where

DR = Density of merge influence area (pc/km/ln)

VR = on-ramp peak 15-min flow rate (pc/h)

$V12$= flow rate entering ramp influence area (pc/h)

LA = Length of acceleration lane (m)

2.2.5.2. Diverge Influence area

Analysis procedures for diverge influence areas follow the same general approach as that for merge areas. Capacity entering the diverge influence area $= V12 = VR + (VF - VR)PFD$

PFD = proportion of through freeway flow remaining in Lanes 1 and 2 immediately upstream of diverge

Table 2.13: Models for predicting V12 at off- ramps

	$V12 = VR + (VF - VR)PFD$ (2-16)
4-lane freeway(2 lanes -each direction)	$P_{FD} = 1.00$ (2-17)
6-lane freeway(3 lanes-each direction)	$P_{FD} = 0.76 - 0.000025V_F - 0.000046V_R$ (2-18) $P_{FD} = 0.717 - 0.000039v_F + 0.184V_U/L_{UP}$ (2-19) $P_{FD} = 0.616 - 0.00021V_F + 0.038V_D/L_{down}$ (2-20)
8-lane freeway(4 lanes-each direction)	$P_{FD} = 0.436$ (2-21)

Determining LOS of diverge area

LOS criteria for diverge areas are based on density in the diverge influence area.

$$DR = 2.642 + 0.0053V12 - 0.0183LD \qquad (2\text{-}22)$$

L_D=length of deceleration lane (m)

2.3. Geometric Features of Partial Cloverleaf Interchange

A variety of partial cloverleaf interchanges can be created with one or two loop ramps. In such cases, one or two of the outer ramps take the form of a diamond ramp, allowing a movement to take place by making a left turn. In some partial cloverleaf configurations, left turns also may be made onto or off of a loop ramp. This section describes the design guidance related to the ramp proper include design speed and horizontal geometrics. (AACRA, 2003).

Exit Geometry

Exit geometry for application to exit ramps and free flow ramps that are one lane width at the nose .The deceleration distance is measured to the end of the queue from the exit ramp intersection or from the target point of any curve on the ramp that requires slowing down. (AACRA, 2003)

Table 2.14: Recommended deceleration Lengths

Highway design speed, kmph	Deceleration length (D), m								
	Exit curve design speed, kmph								
	0	20	30	40	50	60	70	80	90
80	100	95	85	75	60	45	25		
100	155	150	140	130	115	100	80	55	30

Entry Geometry

Entry ramp require sufficient distance to accommodate the level difference between the arterial road and the freeway as well as length for vehicles to accelerate and merge with the freeway traffic

Table 2.15: Recommended Acceleration Lengths *(*AACRA, 2003*)*

Highway design speed, kmph	Acceleration length (Level Grade), m							
	Exit curve design speed, kmph							
	Stop	20	30	40	50	60	70	80
80	235	220	210	195	165	125	75	
100	450	435	425	410	380	340	290	220

Design speed

Because of the different in the type of geometry used for the through carriageway and the interconnecting ramps, it is possible for high relative speeds of vehicles to occur when moving from one element to anther and speeds of vehicles to be carefully considered in the design of the interchange element. The recommended range of operating speeds for various ramp arrangements as shown below. (AACRA, 2003)

Table 2.16: Recommended ramp design speeds

Form of Interchange	Type of Ramp	Range of Freeway Design Speeds (Km/h)		
System		100-120	80-100	60-80
		Range of Ramp Design Speeds (Km/h)		
	Loop	50	40-50	30-40
	Semi-Direct	80-90	70-80	50-70
	Direct	90-100	70-90	60-70
Service	Loop	50	40-50	30-40
	Diagonal Ramps	80-90	70-80	60-70

2.4. Traffic Forecasting

The Project Traffic Forecasting Process estimates traffic conditions used for determining the geometric design of a roadway and/or intersection and is required for reconstruction, adding lanes, bridge replacement, new roadway projects, and major intersection improvements. (FDOT,2002). The calculation of future traffic in urban areas requires the consideration of a greater variety of the following factors

AADT & K_{30}

Annual Average Daily Traffic (AADT):- is the estimate of typical daily traffic on a road segment for all days of the week over the period of one year. AADT is determined by dividing the total volume of traffic on a highway segment for one year by the number of days in the year (FDOT,2002). To forecast future AADT, traffic growth rate will be known**.**

Traffic growth rate: - is used to estimate yearly rate of traffic growth and to estimate future traffic condition. (Sheladia Associates, lnc.USA in Association with Pan Africa Consultants PLc,2003) estimated traffic growth rates of region of Ethiopia from 2011-2025 period.

Table 2.17: Traffic growth rates for 2011-2025 years (Sheladia Associates,2003)

Region	Lcw	Medium	heavy	Articulated trucks	Pass.Car 4WD	Light /heavy bus
Addis Abeba and Dire Dawa	8.5	7.1	7.8	9.9	10.6	7.8
Harer	9.0	7.5	8.2	10.5	11.2	8.2
Benishangul-Gumuz and Ganbela	7.2	6.0	6.6	8.4	9.6	6.6
Oromia	6.6	5.5	6.1	7.7	8.2	6.1
Afar and Sumalia	6.0	5.0	5.5	7.0	7.5	5.5
Amhara and Tigray	5.4	4.5	5.0	6.3	6.7	5.0

Determination of Cumulative Traffic Volumes

In order to determine the cumulative number of vehicles over the design period of the road, the following procedure should be followed (ERA, 2002).

1. Determine the initial traffic volume ($AADT_i$) using the results of the traffic survey and any other recent traffic count information that is available. For paved roads, detail the AADT in terms of car, bus, truck, and truck-trailer.

2. Estimate the annual growth rate "i" expressed as a decimal fraction and the anticipated number of years 'N'. $AADT_N = AADT_i(1 + i)^N$ (2-23)

K and K_{30}

K is the proportion of AADT occurring in an hour. K-Factor is critical in traffic forecasts because it defines the peak hours of road use, typically traffic going to work and coming home. Since this is when the roads will be the most used, it is appropriate to design the system to handle this level of congestion. It is not financially feasible, however, to build for the peak hour of the year, so the 30th highest hour of the year has been chosen as the design hour. K_{30} is the proportion of AADT occurring during the 30th highest hour of the design year. Traffic projections are expressed as AADT and Design Hour Volume (DHV). AADT and DHV are related to each other by the ratio commonly known as K_{30}, as expressed in the equation. (FDOT,2002)

$$DHV = AADT\ x\ K30 \qquad (2\text{-}24)$$

K30 should be measured and not artificially computed using a mathematical equation. However, it is not possible to measure K_{30} at every count site, so the information gathered by the permanent count sites is used to estimate K_{30} when short-term traffic counts are used. The basic assumption is that K_{30} is based on roadway type and land use characteristics and remains relatively constant over time (as long as the roadway type and land use characteristics stay constant). Therefore, an accurate estimate of K_{30} for the current roadway system will be a reasonable estimate of K_{30}

Uganda design manual (2005) recommend that field data should be used to estimate K factors for estimation of DHV and in the absence of such factors the 30th-highest DHV can be estimated by applying 0.15 and 0.10 to ADT for rural highways and urban roads, respectively.

3. Study Area Description

The site study situated is found in Addis Ababa (Gotera interchange), in the previous time before the current interchange was constructed, this junction was very crowded and many problems were happened, but now the road was built by foreign contractors and consultant (chine's company). To construct the current interchange AACRA signed an agreement with Shanghai Construction Group of China at November 10, 2006. Even though it was originally scheduled for 18 months, the work was completed and opened to traffic after 22 months may 1, 2009 with total construction cost of 315 million Birr

This intersection has four legs with Partial Cloverleaf interchange type in the direction of south towards Debre-zeyt (D) in the direction of East towards Bole (B) in the direction of North towards Meskel–Square (M) and in the direction of West towards Kera (K).This interchange is a total –alternative interchange at which two urban arteries intersect. The interchange is the two level total alternative interchange scheme includes the main lines and ramps system, ground road (service road) system and foot path system which are arranged side by side on the ground.

1) M**ain Lines** : the interchange is designed with two levels the E-W main line runs over the rail road and over the S-N main line ; and the S-N main line runs under the E-W main line and then over the rail road .
2) **Ramps**: The ramps of the right –turning traffic are all designed with bound lanes road, the SE, EN, NW ramps are all of the ground roads. The two ramps for left –turning, i.e. ES and WN ramps, 5 are of clover- leaf type ramp ; the ES and WN ramps for left –turning traffic are of twisty ramp and ground road . The ramps are mainly designed with one single –direction lane, but the single direction double –lane ramp is adopted after the WS and ES ramps join together, and the starting section of the EN and ES ramps are designed with single – direction double –lanes.
3) Overall design of service roads : As the interchange is total-alternative interchange , the motor vehicle lanes within the interchange range are enclosed and should not be interfered by pedestrians and non – motor vehicles .In order to provide convenience for the pedestrians, non-motor vehicles and limited height vans within the interchange area , the interchange is designed with a service road system . There are two types of the service roads. Type I is for small vans and non-motor vehicles running in and out on right side and is totally 3.5m wide, a road of grouted asphalt concert. Type II is for double –direction traffic and shared by motor and non motor vehicles and pedestrian; it is 6.0m wide. The service road always runs under the main lines and

ramps; with net wide 6.0m and clear height 2.5m, i.e the small vans /mini-buses with limit height of 2.0-2.2m are allowed to run on the service road.

Figure 3.1: Study site

3.1. Data Collection

Traffic data and Geometric data were required in order to achieve the objective of this thesis. As much as possible, the traffic data collected should indicate the existing peak hour traffic conditions. More or less the geometric data is found in AACRA (Addis Ababa city Road Authority)

3.2. Geometric Design Elements

The following sections describe many of the detailed design elements associated with intersections including approach grades intersection alignment, pavement corner radii, auxiliary lanes, the number and width of lanes, channelization islands, median openings, pedestrian curb ramps and crosswalks, bicycle lane treatments, and bus stop geometric element of the site will be obtained from AACRA and by measurement and which comprise three elements basic free way sections, weaving area and ramp junctions

3.2.1. Plan Design

The plain line type of main lines and ramps within the scope of interchange are all designed with horizontal curves, and the plan design indexes are as below:

1) Main line : circular curve radius > 300m, horizontal curve length 100m and V = 60km/hr
2) Ramps : circular curve radius >45m, horizontal curve length 50m,V=30km/hr

The mainline horizontal curve is designed with transition curve as required and the transition curve length meets the specification, the ramps are designed with transition curve where possible. The main lines of the interchange are of dual –direction four lane roads (the lane width is 3.5 m), locally added with speed up and speed down lanes. Of the main lines, the bridge sections are arranged with 0.5m –wide anti collision intercepted piers and the ground road sections are arranged with 2m –wide intercepted green in the middle of the road. The ground roads connecting the interchange are of dual –direction six lane roads, and a 3.5 m-wide service roadway and 1.5 m-wide footpath on each side of the roads. A 0.75m wide intercepted green is arranged between the main roadway and service roadway, which is in the same width as road shoulder.

The space between entrance and exit of the ramp: exit-exit: ≥80m; exit-iterance :≥ 40m It is designed with single -lane ramps and double –lane ramps, and the entrance and exit are generally arranged in parallel .The WN ramps entrance of the S-N main line is of direct entrance. The circular radius at the end is 0.6m; the speed up section length is 190m, the speed down section length is 65m and the transition section length is 35m.

3.2.2. Longitudinal Section Length

The gradient limit and gradient line limit of the main lines and ramps are specified as follows

1) Longitudinal gradient of main line and ramp is respectively $i<6\%$ and $i<6.5\%$
2) The gradient line of main line of main and ramp is respectively 170m and 85m at minimum.
3) The vertical curve radius of main line and ramp is respectively $R \geq 1000$m and $R \geq 400$m

The design elevation of main line means the road surface elevation at the brims of the central intercepted belt (2m wide) and that of ramp means the road surface elevation at the center line of the ramp.

Table 3.1: Summery of technical criteria adopted in design (taken from AACRA)

Description	Main line	Cloverleaf ramp	By pass ramp	Directional ramp
Designed vehicle speed(km/h)	60	30	30	40
Min. radius without super elevation (m)	600	45	45	80
Min. radius in general(m)	300	/	/	/
Min. length of horizontal curve (m)	100	50	50	65
Min length of transition curve(m)	50	25	25	35
Parking stadia	70	30	30	40
Recommended max. longitudinal slope	5	6	6	6
Min. length of sloping section(m)	170	85	85	110
General min .radius of convex curve	1800	400	400	600
Limited min .radius of convex curve	1200	250	250	400
General min .radius of concave curve	1500	400	400	700
Limited min .radius of concave curve	1000	250	250	450
Minimum length of vertical curve	50	25	30	35
Max. height of filling	6	6	6	6
Max. super elevation transverse slop	2	2	2	2
Net clearance of vehicle under bridge	5.5	5.5	5.5	5.5

3.3. Traffic Data

As it is known Gotera interchange has four legs with main lines and different type of ramp. The ramps are direct ramp, indirect ramp and loop ramps. So, in order to get the exact estimation of capacity of the interchange traffic data is collected at each lane junction of the interchange. Motorized and non-motorized traffic volume has a significant effect on capacity. Because of this motorized and non-motorized volume data is be collected at peak hour (data collection at peak hour each hour is divided in to 15 min time interval and recorded the data for each 15min time interval and take the highest value of the data from the four 15min time interval and change in to hourly volume and the different traffic category into similar values which is passenger car unit by using EPCU) with their direction of movements. The data is collected for three hours (180minute) duration during peak hour. During data collection method the, motorized traffic (vehicle is categorized based on their class) and then current and future condition of the interchange determined using traffic growth rate. Fig 3.2 shows points at which the traffic data is taken.

Traffic movement of vehicle and vehicle's Volume classification are important parameters for capacity analysis. Vehicles counted are summarized as shown in table 3.2. At each direction of the interchange and in order to identify the place at which the data is taken, each place is marked as shown in Fig.3.2

When taking the traffic data

- Select the place where the data is collected
- Prepare the site book or paper which contains all vehicle category
- Arrange persons at the specific position who collected the data
- The data is collected for one peak hour
- Select the peak hour, at morning (1:30-2:30), at midday (6:15-7:15) and at night (11:30-12:30) (since in our country these hours are rush hours because employees are travel from home to office and office to home and traffic congestion is common).
- This one hour is divided to 15 minute
- This one hour is divided to 15 minute
- Data is collected for three peak hours

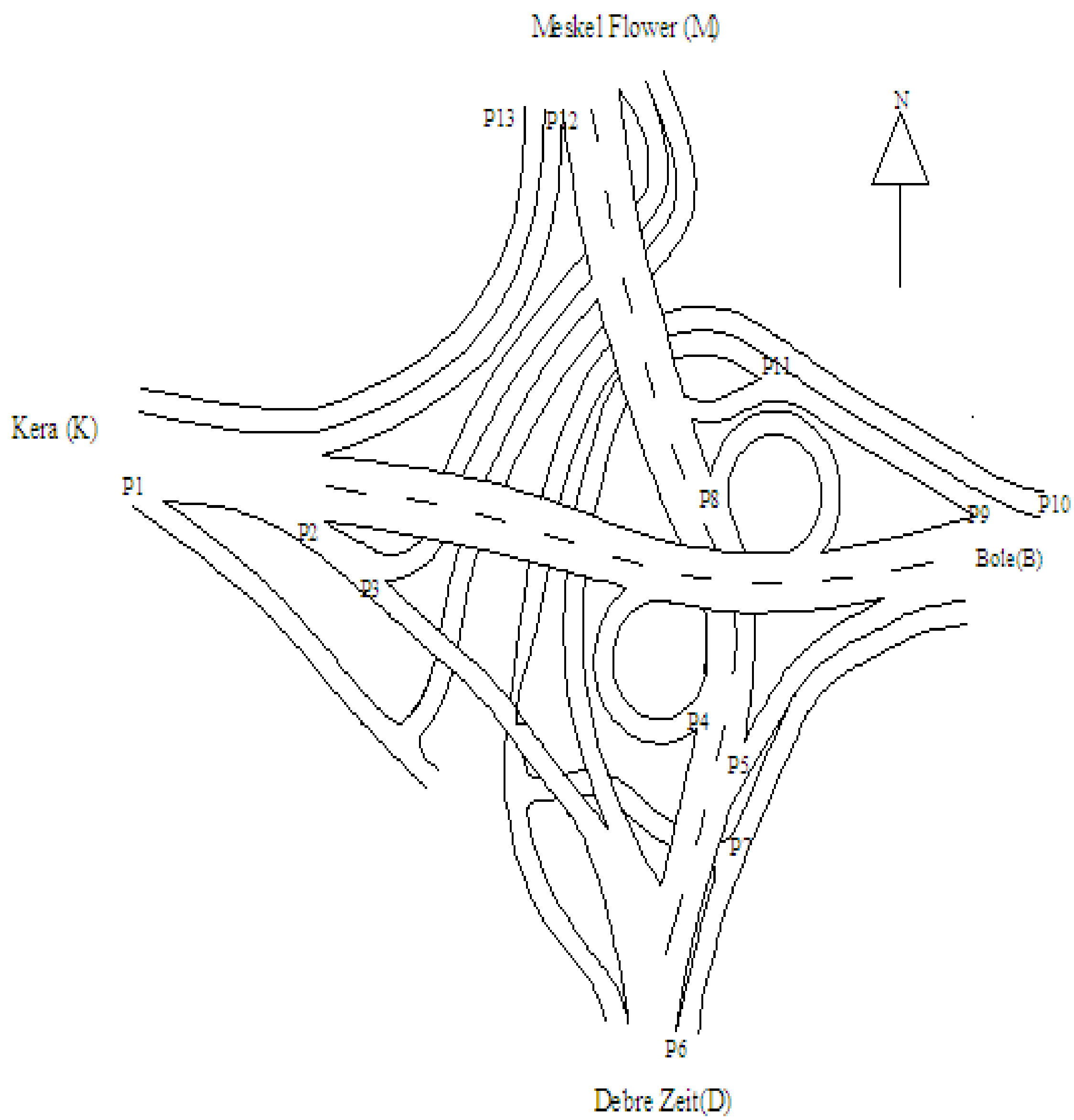

Figure 3.2: Points at which the traffic data is taken

Table 3.2 Collected traffic data at peak hour by their vehicle category (refer to figure 3.2)

No	Assigned Road	Passenger car/4WD	Minibus	Light& heavy bus	Pick up	LCV	Medium Truck	Heavy Truck
1	K-M (P1)	89	24	2	40	34	8	1
2	K-B (p2-P4′)	496	107	3	93	23	2	0
3	K-B (P4′-P5′)	606	117	5	112	24	4	0
4	K-D&M(p2)	499	239	56	101	25	14	6
5	K-M(p3)	347	163	51	81	20	13	0
6	K-D (p3-P3′)	152	76	5	20	5	1	6
7	M-B (p4)	110	10	2	19	1	2	0
8	M-D (p4)	429	220	57	89	11	9	4
9	D-M (p5-P8)	940	274	122	148	72	25	5
10	D-B (p5-P5′)	250	56	9	75	14	6	1
11	D-M&B (p6-P5)	1190	330	131	223	86	31	6
12	D-M (p7)	28	120	-	23	17	0	0
13	D-B (p7)	127	13	18	23	10	4	0
14	D-K (p8-P8′)	223	92	37	36	38		3
15	D-M (p8-P11′)	717	182	85	112	34	25	2
16	B-K (p9-P8′)	650	450	43	150	89	35	2
17	B-K (P8′-P12)	873	542	80	186	127	35	5
18	B-D&M (p9-P11)	499	243	50	124	84	11	2
19	B-D (P10)	61	4	-	17	4	0	0
20	B-M (P11-P11′)	310	230	43	85	65	6	0
21	B-D (P11-P3′)	189	13	7	39	19	5	2
22	M-D (P12-P4)	539	230	59	108	12	11	4
23	M-K (P12-P12′)	350	168	12	78	75	5	0
24	M-K (P13-P13′)	201	25	5	45	13	0	1
25	B&D-M(P11′-P3″1′	1027	412	128	197	99	31	2

Table 3.3: Summarized Hourly vehicle volume at interchange at peak hour

No	Assigned road segment	Heavy vehicles			Light vehicles	Total No. of veh/h
		Buses	Truck and Trailer	Total		
1	K-M (P1)	2	9	11	187	198
2	K-B (p2-P4′)	3	2	5	719	724
3	K-B (P4′-P5′)	5	4	9	859	868
4	K-D&M(p2)	56	20	76	864	940
5	K-M(p3)	51	13	64	611	675
6	K-D (p3-P3′)	5	7	12	253	265
7	M-B (p4)	2	2	4	140	144
8	M-D (p4)	57	13	70	749	819
9	D-M (p5-P8)	122	30	152	1434	1586
10	D-B (p5-P5′)	9	7	16	395	411
11	D-M&B (p6-P5)	131	37	168	1829	1997
12	D-M (p7)	0	0	0	188	188
13	D-B (p7)	18	4	22	173	195
14	D-K (p8-P8′)	37	3	40	389	429
15	D-M (p8-P11′)	85	27	112	1045	1157
16	B-K (p9-P8′)	43	37	80	1339	1419
17	B-K (P8′-P12)	80	40	120	1728	1848
18	B-D&M (p9-P11)	50	13	63	950	1013
19	B-D (P10)	0	0	0	86	86
20	B-M (P11-P11′)	43	6	49	690	739
21	B-D (P11-P3′)	7	7	14	260	274
22	M-D (P12-P4)	59	15	74	889	963
23	M-K (P12-P12′)	12	5	17	671	688
24	M-K (P13-P13′)	5	1	6	284	290
25	B&D-M(P11′-P3″1′	128	33	161	1735	1896

Other data which is observed on site is free flow speed, to determine free flow speed on site the data is observed during low to moderate traffic flow. FFS which is observed on site is as follows

Table 3.4: Free flow speed of interchange of each line

No	Assigned Road	FFS
1	K-M (P1)	60
2	K-B (p2-P4′)	75
3	K-B (P4′-P5′)	70
4	K-D&M(p2)	55
5	K-M(p3)	60
6	K-D (p3-P3′)	70
7	M-B (p4)	42
8	M-D (p4)	70
9	D-M (p5-P8)	68
10	D-B (p5-P5′)	60
11	D-M&B (p6-P5)	70
12	D-M (p7)	40
13	D-B (p7)	50
14	D-K (p8-P8′)	42
15	D-M (p8-P11′)	70
16	B-K (p9-P8′)	70
17	B-K (P8′-P12)	70
18	B-D&M (p9-P11)	50
19	B-D (P10)	50
20	B-M (P11-P11′)	45
21	B-D (P11-P3′)	60
22	M-D (P12-P4)	70
23	M-K (P12-P12′)	45
24	M-K (P13-P13′)	45
25	B&D-M(P11′-P3″1′	70

3.4. Data Analysis

3.4.1. Data Analysis of main lines and ramps

Taking in to consideration all the above geometric and traffic summarized data which are obtained from AACRA and site observation, the capacity and level of service of each lane is precedes using three performance characteristics (Flow rate in passenger, pc/h/ln Average passenger car speed and Density) at current and future condition of the interchange .

As shown from table 3.2 and Figure 3.2 there are 25 points at which traffic data is collected based on their vehicle category (bus, truck and trailers, motor cycle, land cursers ….etc) .and then change all traffic class into cumulative passenger car by using equivalent passenger car unit. For traffic data analysis percentage of heavy vehicles (bus and truck) are essential. The data which is shown table 3.2 converted to passenger car unit and also computed percentage of heavy vehicle as shown below.

Table 3.5: Total number of vehicle and Percentage of heavy vehicle

No	Assigned road segment	Heavy vehicles			Light vehicles	Total No. of veh/h	Percentage of heavy veh.(PT)
		Buses	Truck and	Total			
1	K-M (P1)	2	9	11	187	198	5.56
2	K-B (p2-P4′)	3	2	5	719	724	0.69
3	K-B (P4′-P5′)	5	4	9	859	868	1.04
4	K-D&M(p2)	56	20	76	864	940	8.09
5	K-M(p3)	51	13	64	611	675	9.48
6	K-D (p3-P3′)	5	7	12	253	265	4.53
7	M-B (p4)	2	2	4	140	144	2.78
8	M-D (p4)	57	13	70	749	819	8.55
9	D-M (p5-P8)	122	30	152	1434	1586	9.58
10	D-B (p5-P5′)	9	7	16	395	411	3.89
11	D-M&B (p6-P5)	131	37	168	1829	1997	8.41
12	D-M (p7)	0	0	0	188	188	0.00
13	D-B (p7)	18	4	22	173	195	11.28
14	D-K (p8-P8′)	37	3	40	389	429	9.32
15	D-M (p8-P11′)	85	27	112	1045	1157	9.68
16	B-K (p9-P8′)	43	37	80	1339	1419	5.64
17	B-K (P8′-P12)	80	40	120	1728	1848	6.49
18	B-D&M (p9-P11)	50	13	63	950	1013	6.22
19	B-D (P10)	0	0	0	86	86	0.00
20	B-M (P11-P11′)	43	6	49	690	739	6.63
21	B-D (P11-P3′)	7	7	14	260	274	5.11
22	M-D (P12-P4)	59	15	74	889	963	7.68
23	M-K (P12-P12′)	12	5	17	671	688	2.47
24	M-K (P13-P13′)	5	1	6	284	290	2.07
25	B&D-M(P11′-P3″1′	128	33	161	1735	1896	8.49

The target of this thesis is determining the variable (capacity and LOS) of the interchange currently and to the future, to determine these following variables are determined.

- Flow rate
- Average speed(derived from free flow speed)
- Density

Flow rate (pc/h/ln) -: During traffic data collection at peak hour each hour is divided in to 15 min time interval and recorded the data for each 15min time interval and take the highest value of the data from the four 15min time interval and change in to hourly volume and the different traffic category into similar values which is passenger car unit by using EPCU. From hourly volume and geometric data which is explained in topic three, flow rates is computed.

Calculating the flow rate for a basic freeway sections

$Vp \;\; = V/\,(PHF)(N)(fp)(fHV)$

Where Vp = 15min passenger car equivalent flow rate (pc/h/ln)

V = Hourly peak vehicle volume, veh/h in one direction

PHF = Peak hour factor

N = Number of traffic lanes in one direction

fp = Driver population factor range: 0.85-1.00 use 0.98 for commuter traffic

fHV = Heavy vehicle adjustment factor

$fHV \; = \; 1/\,(1 + PT(ET - 1) + PR(ER - 1))$

Where

PT = Portion of trucks and buses

PR = Portion of recreational vehicles

ET = Passenger car equivalency for trucks

ER = Passenger car equivalency for recreational vehicle

Portion of trucks and buses are computed as shown from table 3.5, portion of recreational vehicle is assume zero, driver population factor, Peak hour factor based on table 2.7 assumptions for freeway mainline and merge/diverge analysis . To determine $ET,$ as shown table 2.6 Suggested passenger car equivalent factors of bus (>4tyers) is 2.25 and Goods (>4tyers) has 2.1 .So, to calculate PCE of the above three take the average

(2.25+2.1)/2=2.17, this is Passenger car equivalents for trucks and buses (ET)

Table 3.6 Summarized Flow Rate at interchange at peak hour

No	Assigned road segment	Traffic veh/h	PT	*ET*	*RV*	fHV=1/(1+ PT(ET-1))	PHF	N	fP	VP
1	K-M (P1)	198	0.06	2.17	0	0.934	0.88	1	0.98	246
2	K-B (p2-P4′)	724	0.01	2.17	"	0.988	0.88	2	0.98	425
3	K-B (P4′-P5′)	868	0.01	2.17	"	0.988	0.88	3	0.98	339
4	K-D&M(p2)	940	0.08	2.17	"	0.914	0.88	2	0.98	596
5	K-M(p3)	675	0.09	2.17	"	0.905	0.88	1	0.98	865
6	K-D (p3-P3′)	265	0.05	2.17	"	0.945	0.88	1	0.98	325
7	M-B (p4)	144	0.03	2.17	"	0.966	0.88	1	0.98	173
8	M-D (p4)	819	0.09	2.17	"	0.905	0.88	2	0.98	525
9	D-M (p5-P8)	1586	0.10	2.17	"	0.895	0.88	2	0.98	1,027
10	D-B (p5-P5′)	411	0.04	2.17	"	0.955	0.88	1	0.98	499
11	D-M&B (p6-P5)	1997	0.08	2.17	"	0.914	0.88	3	0.98	844
12	D-M (p7)	188	0.00	2.17	"	1.000	0.88	1	0.98	218
13	D-B (p7)	195	0.11	2.17	"	0.886	0.88	1	0.98	255
14	D-K (p8-P8′)	429	0.09	2.17	"	0.905	0.88	1	0.98	550
15	D-M (p8-P11′)	1157	0.10	2.17	"	0.895	0.88	2	0.98	749
16	B-K (p9-P8′)	1419	0.06	2.17	"	0.934	0.88	2	0.98	880
17	B-K (P8′-P12)	1848	0.06	2.17	"	0.934	0.88	3	0.98	764
18	B-D&M (p9-P11)	1013	0.06	2.17	"	0.934	0.88	2	0.98	629
19	B-D (P10)	86	0.00	2.17	"	1.000	0.88	1	0.98	100
20	B-M (P11-P11′)	739	0.07	2.17	"	0.924	0.88	2	0.98	464
21	B-D (P11-P3′)	274	0.05	2.17	"	0.945	0.88	2	0.98	168
22	M-D (P12-P4)	963	0.08	2.17	"	0.914	0.88	3	0.98	407
23	M-K (P12-P12′)	688	0.02	2.17	"	0.977	0.88	2	0.98	408
24	M-K (P13-P13′)	290	0.02	2.17	"	0.977	0.88	1	0.98	344
25	B&D-M(P11′-P3″1′	1896	0.08	2.17	"	0.914	0.88	3	0.98	801

Average passenger car speed (km/h)-: Mean speed of passenger cars can be computed as

For 90$\leq FFS \leq$120 and flow rate (Vp)

(3100-15FFS)<$Vp \leq$ (1800+5FFS)

$$S = FFS - \left[1/28(23FFS - 1800) \left(\frac{vp + 15FFS - 3100}{20FFS - 1300} \right)^{2.6} \right]$$

For 90$\leq FFS \leq$120 and

Vp< (3100-15FFS), $S = FFS$

Free-flow speed (FFS): **-** the free-flow speed of a main line of freeway can be determined on site during low to moderate traffic flow or can be estimated as follows based on HCM

$$FFS = BFFS - fLW - fLC - fN - fID$$

Where FFS = free-flow speed of the freeway, km/h

$BFFS$ = base free-flow speed of the freeway (110km/h for urban and suburban freeways, 120km/h for rural freeways) for only main lines.

fLW = adjustment for lane width, km/h

fLC =adjustment for lateral clearance km/h

fN =adjustment for number of lanes, km/h

fID = adjustment for interchange density, km/h

From geometric data main line of lane width is 3.5 it's lane adjustment factor is 1, shoulder lateral clearance is 0.75 and from table 2.3 adjustment factor of lateral clearance =3.4, to compute free flow speed of main line of freeway interchange adjustment of number of lanes and interchange density adjustment also required, table 2.5 and 2.4 shows number of lane and interchange interval and their corresponding adjustment factors

Table 3.7 Factors which affect free flow speed

No	Factors	Values for factors	Corresponding adjustment factors
1	Lane width	3.5	1.0
2	Shoulder lateral clearance	0.75	3.4
3	No of lanes	2.0	7.3
4	Interchange density per km	$\leq$ 0.3	0.0

$$\mathrm{FFS} = \mathrm{BFFS} - \mathrm{fLW} - \mathrm{fLC} - \mathrm{fN} - \mathrm{fID}$$

fLW =1, fLC =3.4, fN = 7.3, fID =0

FFS = 110-1- 3.4-7.3-0 = 98.3km/hr (for main line)

For 90$\leq FFS \leq$120 and flow rate (Vp)

(3100-15FFS) < $Vp \leq$(1800+5FFS)

$$S = FFS-[1/28(23FFS-1800)\left(\frac{vp+15FFS-3100}{20FFS-1300}\right)^{2.6}]$$

For 90$\leq FFS \leq$120 and

Vp < (3100-15FFS), $S = FFS$

Since FFS which is calculated based on HCM is more than FFS of each line which is measured at site for determination of LOS of the line FFS as shown table 3.4 is used

The average of all passenger-car speeds measured in the field under low- to moderate- volume conditions can be used directly as the FFS of the freeway segment.

3.4.2. Data Analysis of Merge and Diverge Influence Area

3.4.2.1. Analysis of merge influence area

From literature review factors which affect merge influence area are total flow approaching merge area (VF), total ramp flow (VR), total length of acceleration (L_A) and maximum total flow entering the ramp influence area (V_{R12}). All of the factors except L_A are determined from traffic data and L_A taken from geometric data source of AACRA which is 190m for merge areas.

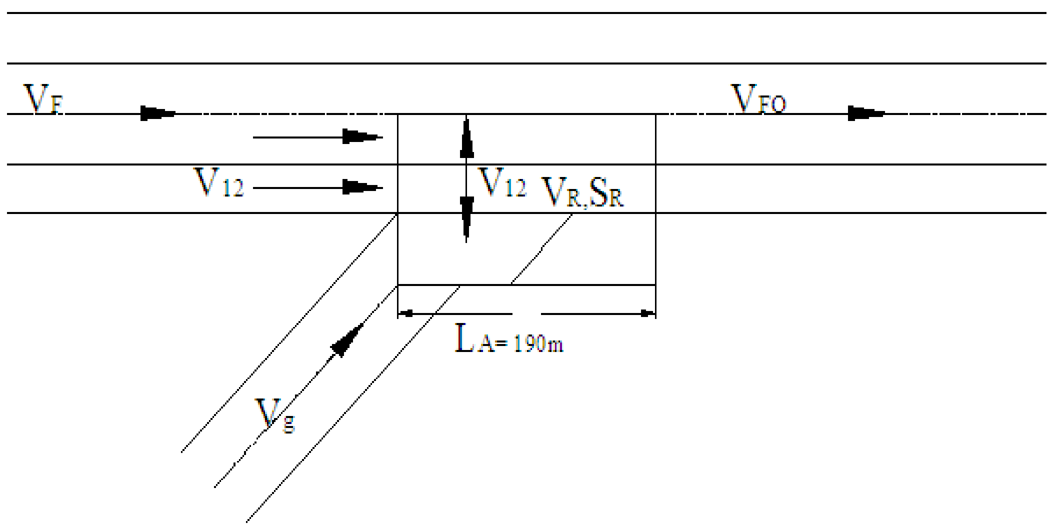

Figure 3.3: Critical merge influence area

To determine level of service of the interchange, AACRA Geometric design manual used as reference of Highway capacity manual (TRB, 2000). (Refer AACRA Geometric design manual, section 16, page 20).

V_{12} is predicted by $V12 = VF * PFM$ as shown table 2.13, P_{FM} value depend on number of lanes Where number of lanes are 4 in both direction (2 in one direction) P_{FM} =1 this implies that V_{12}=V_F, three lanes in each direction $PFM = 0.5775 + 0.000092LA$ and four lanes in each direction $PFM = 0.2178 - 0.000125VR + 0.05887LA/SFR$ and the total flow of at merge influence area is the sum of ramp flow and approaching merge area, $VR12 = V12 + VR$.

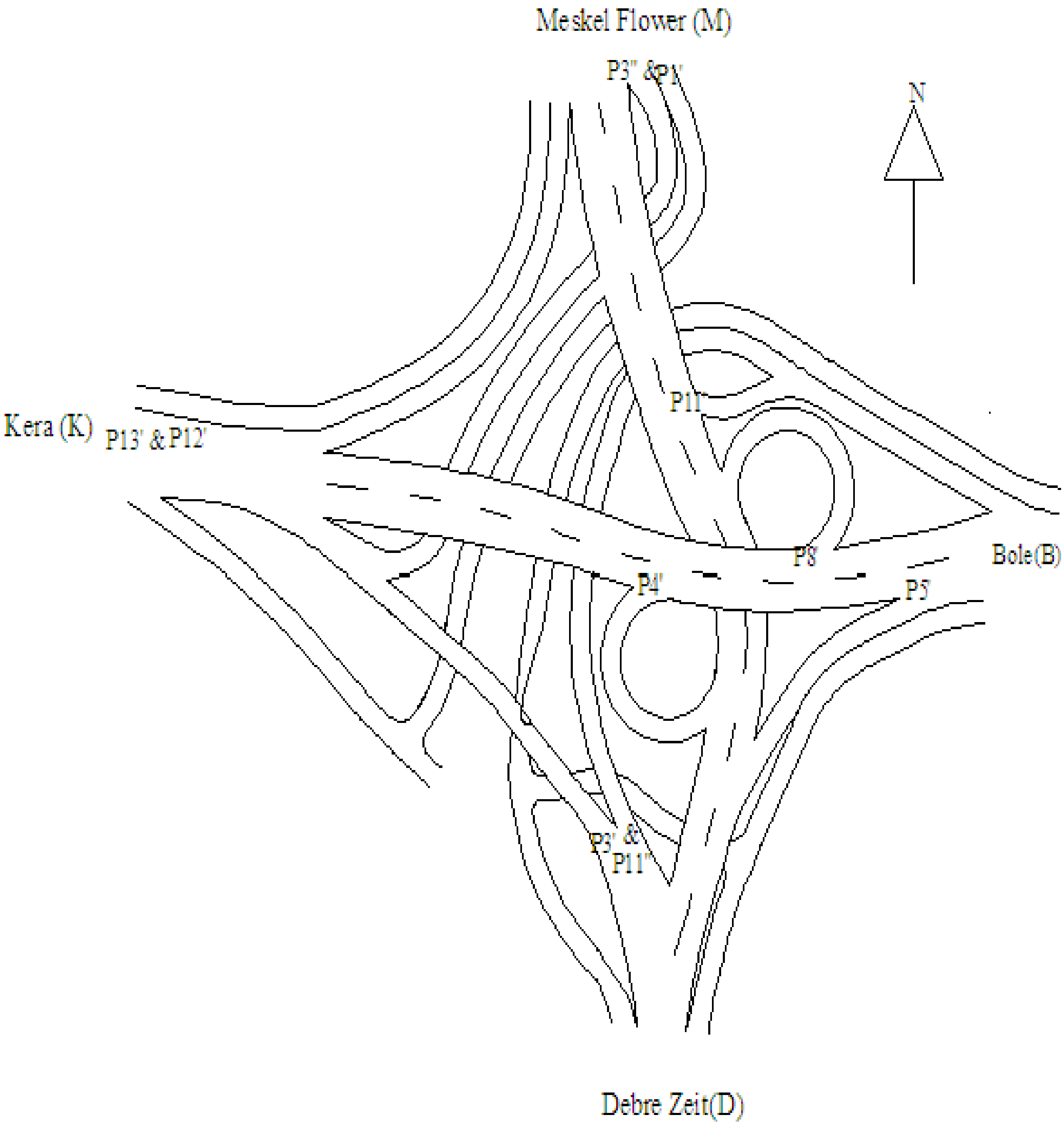

Figure 3.4: Points of merge influence area

3.4.2.2. Analysis of diverge influence area

Diverge influence area capacity and LOS are evaluated as the same as merge influence area ,it is depend on maximum total flow approaching diverge area(V_F), Maximum total flow departing from diverge area, maximum total flow entering the ramp (V_{12}), maximum flow on a ramp (V_R), and geometrically it depends on length of deceleration lane(L_D)

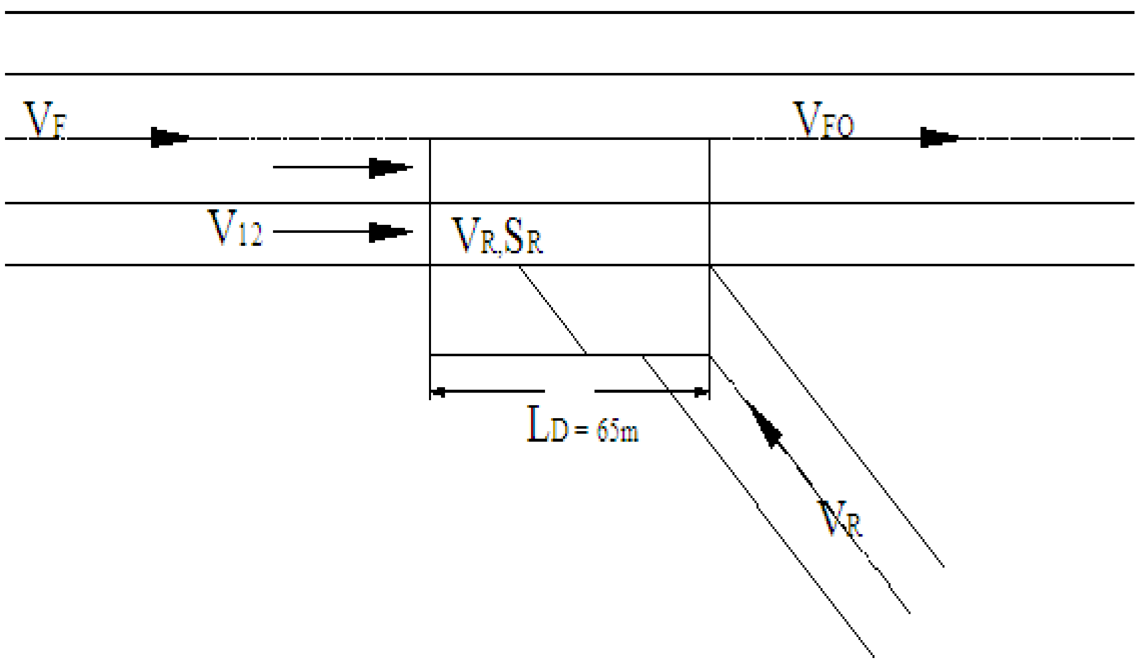

Figure 3.5: Critical Diverge Influences area

For diverge influence area $V12 = VR + (VF - VR)\,PFD$

For 4-lane freeways (2 lane each direction P_{FD} =1.00, for six lane (3-lane each direction)

$PFD = 0.760 - 0.000025VF - 0.000046VR,$ for 8 lane (4 each direction) PFD =0.436.

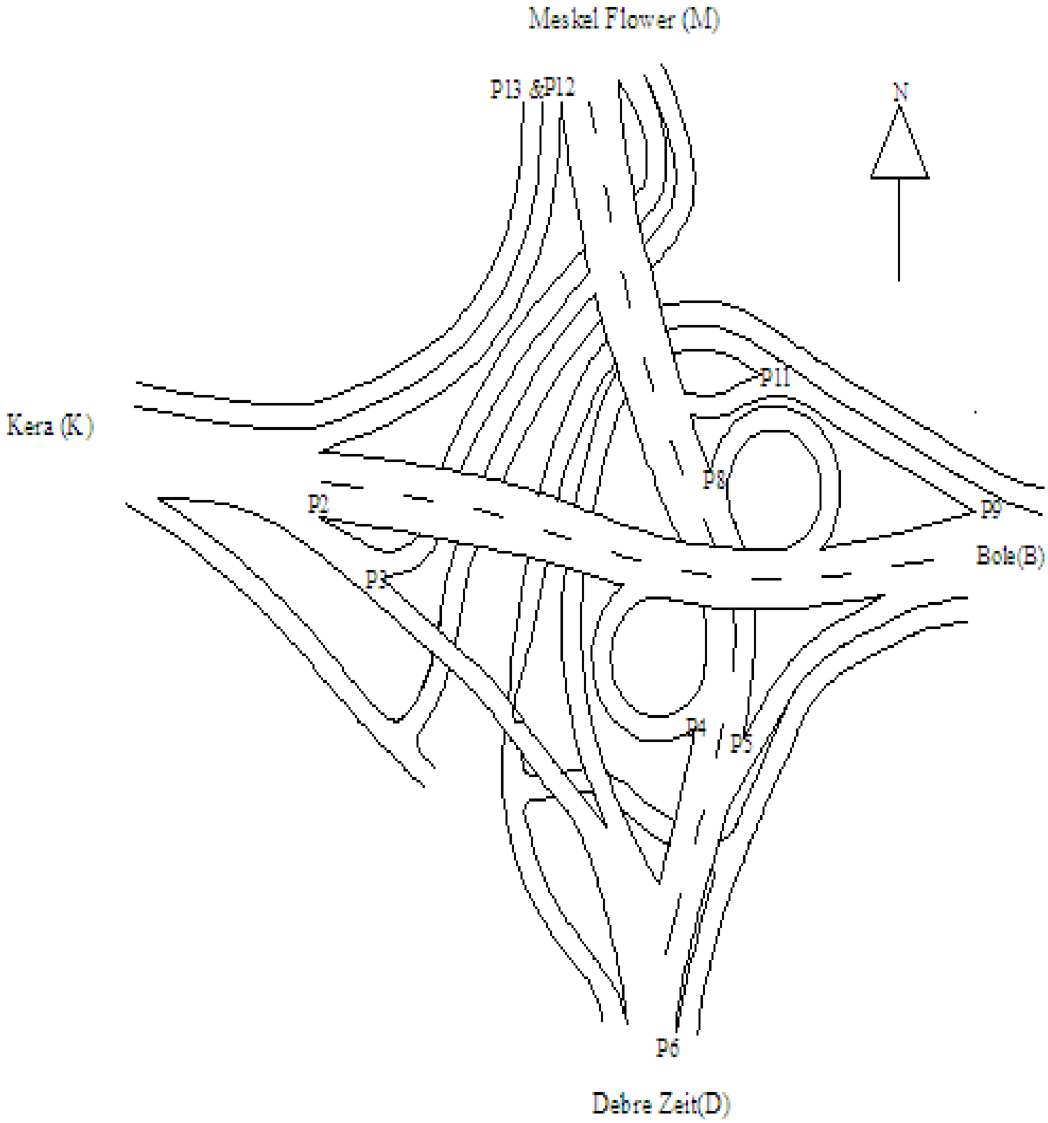

Figure 3.6: Points of Diverge influence area

4.0. Data Results and Discussion

4.1. Main lines and Ramps

Capacity and LOS analysis has been summarized in table 4.1.The performance is measured with density (Vp/s) or level of service according to HCM. From the analysis results , as shown table 4.1 maximum flow rates is obtained from analysis part table 3.6 which is the maximum hourly flow rate during peak traffic flow for each segment . As discussed above free flow speed is calculated based on highway capacity manual , the maximum value is 98.3 km /hr occurred at main lines and the minimum value is 40 km/hr at loop ramps .But free flow speed is measured in site during low to moderate condition traffic flow , the result is as shown table 3.4 from which the maximum value was recorded at main lines designated by K-B(P2-P4’) has 75km/hr. and the minimum value was recorded at loop ramps which is 42km/hr So , it is better to that taking the actual measured value.

To determine the level of service which is expressed by average speed and maximum flow rate that means the amount of density defines the level of service of the road. From table 4.1 the segment which flow from Kera to Meskel-Square designated by K-M(P3) has a maximum density value of 16.8 from 25 segments of the interchange, the second denser segment is Debre-Zeyt to Meskel -Square designated by D-M(P5-P8) has density value of 15.3 and the third denser segment is Debre-Zeyt to Kera which has density value of 13.8. To the contrary from 25 main and ramp lines Bole to Debre- Zeyt designated by B-D(P10) has a least density value that is 2.0 , the second segment is Bole to Debre -Zeyt designated by B-D(P11-P3’) has density values 2.9 the third segment is Kera to Meskel- Squred , K-M(P1) has 4.1 and 246 pc/h/ln.

Generally, from 25 interchange segments 12 lines have level of service of A, 7 lines have level of service B and the rest 6 lines have level of service C.

Table 4.1 Average car speed, Density and LOS of each line

No	Assigned road segment	VP	FFS(km/h)	S	D=Vp/s	LOS
1	K-M (P1)	246	60	60	4.1	A
2	K-B (p2-P4′)	425	75	75	5.7	A
3	K-B (P4′-P5′)	339	70	70	4.8	A
4	K-D&M(p2)	596	55	55	10.8	B
5	K-M(p3)	865	60	60	14.4	C
6	K-D (p3-P3′)	325	70	70	4.6	A
7	M-B (p4)	173	42	42	4.1	A
8	M-D (p4)	525	70	70	7.5	B
9	D-M (p5-P8)	1,027	68	68	15.1	C
10	D-B (p5-P5′)	499	60	60	8.3	B
11	D-M&B (p6-P5)	844	70	70	12.1	C
12	D-M (p7)	218	40	40	5.5	A
13	D-B (p7)	255	50	50	5.1	A
14	D-K (p8-P8′)	550	42	42	13.1	C
15	D-M (p8-P11′)	749	70	70	10.7	B
16	B-K (p9-P8′)	880	70	70	12.6	C
17	B-K (P8′-P12)	764	70	70	10.9	B
18	B-D&M (p9-P11)	629	50	50	12.6	C
19	B-D (P10)	100	50	50	2.0	A
20	B-M (P11-P11′)	464	45	45	10.3	B
21	B-D (P11-P3′)	168	60	60	2.8	A
22	M-D (P12-P4)	407	70	70	5.8	A
23	M-K (P12-P12′)	408	45	45	9.1	A
24	M-K (P13-P13′)	344	45	45	7.6	A
25	B&D-M(P11′-P3″1′	801	70	70	11.4	B

4.2. Results of merge and diverge influence area

4.2.1. Merge influence area

The capacity and LOS analysis results of the merge influence area of the interchange are summarized in table 4.2 and 4.3 respectively.

Table 4.2: Capacity of Merge influence area

No.	Junction	VF	N	LA	PFM	$V12 = VF * PFM$	VR	Capacity of merge influence area $VR12 = V12 + VR$
1	Exit of p4′	850	3	190	0.595	506	173	679
2	Exit of p8′	1760	3	190	0.595	1047	550	1597
3	Exit of p5′	1017	3	190	0.595	605	499	1104
4	Exit of p3′andp11′	1050	2	190	1	1050	661	1711
5	Exit of p3″and p1′	2403	3	190	0.595	1429	1111	2540
6	Exit of p12′andp13′	2292	3	190	0.595	1363	1160	2523
7	Exit of p11′	1498	2	190	1	1498	928	2426

LOS criteria for merge areas are based on density in the merge influence area.

$$DR = 3.402 + 0.00456VR + 0.0048V12 - 0.01278LA$$

Table 4.3: LOS of merge influence area

No.	Junction	LA	$V12$	VR	$D_R = 3.402 + 0.00456VR + 0.0048V12 - 0.01278LA$	LOS
1	Exit of p4′	190	506	173	4	A
2	Exit of p8′	190	1047	550	8.5	B
3	Exit of p5′	190	605	499	6	A
4	Exit of p3′andp11′	190	1050	661	9	B
5	Exit of p3″and p1′	190	1429	1111	12.89	C
6	Exit of p12′andp13′	190	1363	1160	12.8	C
7	Exit of p11′	190	1498	928	12.4	C

Based on table 4.2 from seven merge influence area, Exit of p3″and p1′, Exit of p12′andp13′, Exit of p11′, have 2540, 2523 and 2426 pcuph respectively. This table shows that there is more traffic flow of exit of Meskel squre and kera compared to others. When we see the level of service of (LOS) these points are ‘C’ which shows that more crowded than others, but this value ‘C’ shows that stable flow not free flow.

4.2.2. Diverge influence area

Diverge influence area capacity and LOS are evaluated as the same as merge influence area ,it is depend on maximum total flow approaching diverge area(V_F), Maximum total flow departing from diverge area, maximum total flow entering the ramp (V_{12}), maximum flow on a ramp (V_R), and geometrically it depends on length of deceleration lane(L_D)

Table 4.4: Capacity of Diverge influence area

No	Junction	VF	N	P_{FD}	V_R	Capacity $V12 = VR + (VF - VR)\ PFD$
1	Entrance of p4	1221	2	1	173	1221
2	Entrance of p6	3005	4	0.436	473	1577
3	Entrance of p8	2054	2	1	550	2054
4	Entrance of p9	3018	3	0.62	1258	2349
5	Entrance of p5	2532	3	0.67	499	1861
6	Entrance of p2	2034	2	1	1184	2034
7	Entrance of p3	1184	2	1	865	1184
8	Entrance of p11	1258	2	1	336	1258
9	Entrance of p12	2381	3	0.64	1160	1941

Generally from the above merge and diverge influence area capacity analysis, the total capacity of interchange is 7901 pc/h.

LOS criteria for diverge influence areas are based on density in the area.

$$DR = 2.642 + 0.0053V12 - 0.0183LD$$

Table 4.5: LOS of Diverge influence area

No.	Junction	LD	$V12$	$DR = 2.642 + 0.0053V12 - 0.0183LD$	LOS
1	Entrance of p4	65	1221	7.9	B
2	Entrance of p6	65	1577	9.8	B
3	Entrance of p8	65	2054	12.3	C
4	Entrance of p9	65	2349	13.9	C
5	Entrance of p5	65	1861	11.3	B
6	Entrance of p2	65	2034	12.2	C
7	Entrance of p3	65	1184	7.7	B
8	Entrance of p11	65	1258	8.1	B
9	Entrance of p12	65	1941	11.7	B

5.0. Traffic Forecasting of the Interchange

Traffic Forecasting estimates traffic conditions used for determining the geometric design of a roadway or intersection will be subjected to over the design life. As discussed in literature review it requires the consideration of a greater variety of factors such as AADT, K30 and DHV.

In this portion, future 10 years of traffic flow of the interchange will be forecasted. Because most of the analysis period of flexible pavement is 15-20 years. So, the interchange was opened to traffic in 2006 which is almost nine years and from both past and future condition of the interchange , it is suggested that the project is reliable or not .

As expressed in the literature review, $DHV = AADT\ x\ K30$

DHV (design hourly volume = Vp) so in order to convert Vp to $AADT$i which is the current AADTi of the interchange.

$$Vp = AADT0 \text{ x K30}$$

Table 5.1: Current AADT of each vehicle class ($AADT = V/K30$)

	Assigned road segment	Pas.car/4wd		Minibus		Light &heavy bus		Lcv		Pick up		Med.truck		heavy truck	
		V	AADT	V	$AADT$	V	AADT	V	AADT	V	AADT	V	AADT	V	AADT
1	(P1)	89	890	24	240	2	20	40	400	34	340	8	80	1	10
2	p2-P4’)	496	4960	107	1070	3	30	93	930	23	230	2	20	0	0
3	(P4’-P5’)	606	6060	117	1170	5	50	112	1120	24	240	4	40	0	0
4	(p2)	499	4990	239	2390	56	560	101	1010	25	250	14	140	6	60
5	(p3)	347	3470	163	1630	51	510	81	810	20	200	13	130	0	0
6	(p3-P3’)	152	1520	76	760	5	50	20	200	5	50	1	10	6	60
7	(p4)	110	1100	10	100	2	20	19	190	1	10	2	20	0	0
8	(p4)	429	4290	220	2200	57	570	89	890	11	110	9	90	4	40
9	(p5-P8)	940	9400	274	2740	122	1220	148	1480	72	720	25	250	5	50
10	(p5-P5’)	250	2500	56	560	9	90	75	750	14	140	6	60	1	10
11	(p6-P5)	1190	11900	330	3300	131	1310	223	2230	86	860	31	310	6	60
12	(p7)	28	280	120	1200	-		23	230	17	170	0	0	0	0
13	(p7)	127	1270	13	130	18	180	23	230	10	100	4	40	0	0
14	p8-P8’)	223	2230	92	920	37	370	36	360	38	380		0	3	30
15	p8-P11’)	717	7170	182	1820	85	850	112	1120	34	340	25	250	2	20
16	(p9-P8’)	650	6500	450	4500	43	430	150	1500	89	890	35	350	2	20
17	(P8’-P12)	873	8730	542	5420	80	800	186	1860	127	1270	35	350	5	50
18	(p9-P11)	499	4990	243	2430	50	500	124	1240	84	840	11	110	2	20
19	(P10)	61	610	4	40	-		17	170	4	40	0	0	0	0
20	(p11-P11’)	310	3100	230	2300	43	430	85	850	65	650	6	60	0	0
21	(p11-P3’)	189	1890	13	130	7	70	39	390	19	190	5	50	2	20
22	(p12-P4)	539	5390	230	2300	59	590	108	1080	12	120	11	110	4	40
23	(p12-P12’)	350	3500	168	1680	12	120	78	780	75	750	5	50	0	0
24	(p13)	201	2010	25	250	5	50	45	450	13	130	0	0	1	10
25	P11’-P3’1’	1027	10270	412	4120	128	1280	197	1970	99	990	31	310	2	20

Table 5.2: Total Current AADT of the interchange for each direction

No	Assigned road segment	Traffic veh/h (V)	N	PT	K30		$AADT0 = \frac{V}{K30}$
1	K-M (P1)	198	1	5.56	0.1		1980
2	K-B (p2-P4′)	724	2	0.69	0.1		7240
3	K-B (P4′-P5′)	868	3	1.04	0.1		8680
4	K-D&M(p2)	940	2	8.09	0.1		9400
5	K-M(p3)	675	1	9.48	0.1		6750
6	K-D (p3-P3′)	265	1	4.53	0.1		2650
7	M-B (p4)	144	1	2.78	0.1		1440
8	M-D (p4)	819	2	8.55	0.1		8190
9	D-M (p5-P8)	1586	2	9.58	0.1		15860
10	D-B (p5-P5′)	411	1	3.89	0.1		4110
11	D-M&B (p6-P5)	1997	3	8.41	0.1		19970
12	D-M (p7)	188	1	0.00	0.1		1880
13	D-B (p7)	195	1	11.28	0.1		1950
14	D-K (p8-P8′)	429	1	9.32	0.1		4290
15	D-M (p8-P11′)	1157	2	9.68	0.1		11570
16	B-K (p9-P8′)	1419	2	5.64	0.1		14190
17	B-K (P8′-P12)	1848	3	6.49	0.1		18480
18	B-D&M (p9-P11)	1013	2	6.22	0.1		10130
19	B-D (P10)	86	1	0.00	0.1		860
20	B-M (P11-P11′)	739	2	6.63	0.1		7390
21	B-D (P11-P3′)	274	2	5.11	0.1		2740
22	M-D (P12-P4)	963	2	7.68	0.1		9630
23	M-K (P12-P12′)	688	2	2.47	0.1		6880
24	M-K (P13-P13′)	290	1	2.07	0.1		2900
25	B&D-M(P11′-P3″1′)	1896	3	8.49	0.1		18960

From the current traffic condition of the interchange, the future traffic condition will be determined using traffic growth rate (i).As shown table 5.2 current AADT of each vehicle class are determined and from this future AADT will be determined**.**

$$AADT_N = AADT\ (1 + i)^N$$

AADT10 = Average Annual Daily Traffic after 10 years

AADT = Current Average Annual Daily Traffic

i = traffic growth rate which described in table 2.17

N = number of years =10 years

Table 5.3: AADT of each mainline and ramp after 10 years

	Assigned road segment	Pas.car/4 wd	minibus	Light &heavy bus	Lcv	Pick up	Med.truck	heavy truck
		$(1+i)^x$ =2.74	$(1+i)^x$ =2.12	$(1+i)^x$ =2.12	$(1+i)^x$ =2.26	$(1+i)^x$ =2.26	$(1+i)^x$ =1.98	$(1+i)^x$ =2.12
		AADT	AADT	AADT	AADT	AADT	AADT	AADT
1	K-M (P1)	2,439	509	42	904	768	158	21
2	K-B (p2-P4′)	13,590	2,268	64	2102	520	40	0
3	K-B (P4′-P5′)	16,604	2,480	106	2531	542	79	0
4	K-D&M(p2)	13,673	5,067	1187	2283	565	277	127
5	K-M(p3)	9,508	3,456	1081	1831	452	257	0
6	K-D (p3-P3′)	4,165	1,611	106	452	113	20	127
7	M-B (p4)	3,014	212	42	429	23	40	0
8	M-D (p4)	11,755	4,664	1208	2011	249	178	85
9	D-M (p5-P8)	25,756	5,809	2586	3345	1627	495	106
10	D-B (p5-P5′)	6,850	1,187	191	1695	316	119	21
11	D-M&B (p6-P5)	32,606	6,996	2777	5040	1944	614	127
12	D-M (p7)	767	2,544	0	520	384	0	0
13	D-B (p7)	3,480	276	382	520	226	79	0
14	D-K (p8-P8′)	6,110	1,950	784	814	859	0	64
15	D-M (p8-P11′)	19,646	3,858	1802	2531	768	495	42
16	B-K (p9-P8′)	17,810	9,540	912	3390	2011	693	42
17	B-K (P8′-P12)	23,920	11,490	1696	4204	2870	693	106
18	B-D&M (p9-P11)	13,673	5,152	1060	2802	1898	218	42
19	B-D (P10)	1,671	85	0	384	90	0	0
20	B-M (P11-P11′)	8,494	4,876	912	1921	1469	119	0
21	B-D (P11-P3′)	5,179	276	148	881	429	99	42
22	M-D (P12-P4)	14,769	4,876	1251	2441	271	218	85
23	M-K (P12-P12′)	9,590	3,562	254	1763	1695	99	0
24	M-K (P13-P13′)	5,507	530	106	1017	294	0	21
25	B&D-M(P11′-P3″1′	28,140	8,734	2714	4452	2237	614	42

5.1. Future LOS and capacity of the interchange

To determine LOS and capacity Vp (flow rate) and average speed of the interchange will be computed from the given AADT10 peak hourly volume (V) =AADT*K30. After computed V flow rate (Vp), average speed and density will be determined and then LOS and capacity of the interchange can be evaluated.

Table 5.4: Traffic volume of each line after 10 years

	Assigned road segment	Passenger car/ 4wd	Minibus	Light & heavy bus	Lcv	Pick up	Medium truck	heavy truck	Total V
1	K-M (P1)	244	51	4	90	77	16	2	484
2	K-B (p2-P4′)	1,359	227	6	210	52	4	0	1,858
3	K-B (P4′-P5′)	1,660	248	11	253	54	8	0	2,234
4	K-D&M(p2)	1,367	507	119	228	57	28	13	2,318
5	K-M(p3)	951	346	108	183	45	26	0	1,658
6	K-D (p3-P3′)	416	161	11	45	11	2	13	659
7	M-B (p4)	301	21	4	43	2	4	0	376
8	M-D (p4)	1,175	466	121	201	25	18	8	2,015
9	D-M (p5-P8)	2,576	581	259	334	163	50	11	3,972
10	D-B (p5-P5′)	685	119	19	170	32	12	2	1,038
11	D-M&B (p6-P5)	3,261	700	278	504	194	61	13	5,010
12	D-M (p7)	77	254	0	52	38	0	0	422
13	D-B (p7)	348	28	38	52	23	8	0	496
14	D-K (p8-P8′)	611	195	78	81	86	0	6	1,058
15	D-M (p8-P11′)	1,965	386	180	253	77	50	4	2,914
16	B-K (p9-P8′)	1,781	954	91	339	201	69	4	3,440
17	B-K (P8′-P12)	2,392	1,149	170	420	287	69	11	4,498
18	B-D&M (p9-P11)	1,367	515	106	280	190	22	4	2,485
19	B-D (P10)	167	8	0	38	9	0	0	223
20	B-M (P11-P11′)	849	488	91	192	147	12	0	1,779
21	B-D (P11-P3′)	518	28	15	88	43	10	4	705
22	M-D (P12-P4)	1,477	488	125	244	27	22	8	2,391
23	M-K (P12-P12′)	959	356	25	176	170	10	0	1,696
24	M-K (P13-P13′)	551	53	11	102	29	0	2	748
25	B&D-M(P11′-P3″1′	2,814	873	271	445	224	61	4	4,693

Table 5.5: Total traffic and percentage of heavy vehicles (PT)

	Assigned road segment	Heavy vehicle			Light vehicles	Total V	PT
		Buses	Truck &trailer	Total			
1	K-M (P1)	4	18	22	462	484	4.59
2	K-B (p2-P4′)	6	4	10	1,848	1,858	0.56
3	K-B (P4′-P5′)	11	8	19	2,216	2,234	0.83
4	K-D&M(p2)	119	40	159	2,159	2,318	6.87
5	K-M(p3)	108	26	134	1,525	1,658	8.07
6	K-D (p3-P3′)	11	15	25	634	659	3.84
7	M-B (p4)	4	4	8	368	376	2.18
8	M-D (p4)	121	26	147	1,868	2,015	7.30
9	D-M (p5-P8)	259	60	319	3,654	3,972	8.02
10	D-B (p5-P5′)	19	14	33	1,005	1,038	3.19
11	D-M&B (p6-P5)	278	74	352	4,659	5,010	7.02
12	D-M (p7)	0	0	0	422	422	-
13	D-B (p7)	38	8	46	450	496	9.29
14	D-K (p8-P8′)	78	6	85	973	1,058	8.01
15	D-M (p8-P11′)	180	54	234	2,680	2,914	8.03
16	B-K (p9-P8′)	91	74	165	3,275	3,440	4.79
17	B-K (P8′-P12)	170	80	250	4,248	4,498	5.55
18	B-D&M (p9-P11)	106	26	132	2,353	2,485	5.31
19	B-D (P10)	0	0	0	223	223	-
20	B-M (P11-P11′)	91	12	103	1,676	1,779	5.79
21	B-D (P11-P3′)	15	14	29	677	705	4.11
22	M-D (P12-P4)	125	30	155	2,236	2,391	6.50
23	M-K (P12-P12′)	25	10	35	1,661	1,696	2.08
24	M-K (P13-P13′)	11	2	13	735	748	1.70
25	B&D-M(P11′-P3″1′	271	66	337	4,356	4,693	7.18

Table 5.6: Future Flow Rate of the interchange

No	Assigned road segment	Traffic veh/h	PT	ET	RV	fHV	PHF	N	fP	VP
1	K-M (P1)	484	4.59	2.17	0	0.95	0.88	1	0.98	591
2	K-B (p2-P4′)	1858	0.56	2.17	“	0.99	0.88	2	0.98	1084
3	K-B (P4′-P5′)	2234	0.83	2.17	“	0.99	0.88	3	0.98	872
4	K-D&M(p2)	2318	6.87	2.17	“	0.93	0.88	2	0.98	1452
5	K-M(p3)	1658	8.07	2.17	“	0.91	0.88	1	0.98	2104
6	K-D (p3-P3′)	659	3.84	2.17	“	0.96	0.88	1	0.98	798
7	M-B (p4)	376	2.18	2.17	“	0.98	0.88	1	0.98	447
8	M-D (p4)	2015	7.30	2.17	“	0.92	0.88	2	0.98	1268
9	D-M (p5-P8)	3972	8.02	2.17	“	0.91	0.88	2	0.98	2519
10	D-B (p5-P5′)	1038	3.19	2.17	“	0.96	0.88	1	0.98	1249
11	D-M&B (p6-P5)	5010	7.02	2.17	“	0.92	0.88	3	0.98	2096
12	D-M (p7)	422	0.00	2.17	“	1.00	0.88	1	0.98	489
13	D-B (p7)	496	9.29	2.17	“	0.90	0.88	1	0.98	638
14	D-K (p8-P8′)	1058	8.01	2.17	“	0.91	0.88	1	0.98	1342
15	D-M (p8-P11′)	2914	8.03	2.17	“	0.91	0.88	2	0.98	1848
16	B-K (p9-P8′)	3440	4.79	2.17	“	0.95	0.88	2	0.98	2106
17	B-K (P8′-P12’)	4498	5.55	2.17	“	0.94	0.88	3	0.98	1851
18	B-D&M (p9-P11)	2485	5.31	2.17	“	0.94	0.88	2	0.98	1530
19	B-D (P10)	223	0.00	2.17	“	1.00	0.88	1	0.98	259
20	B-M (P11-P11′)	1779	5.79	2.17	“	0.94	0.88	2	0.98	1101
21	B-D (P11-P3′)	705	4.11	2.17	“	0.95	0.88	2	0.98	428
22	M-D (P12-P4)	2391	6.50	2.17	“	0.93	0.88	2	0.98	1492
23	M-K (P12-P12′)	1696	2.08	2.17	“	0.98	0.88	2	0.98	1007
24	M-K (P13-P13′)	748	1.70	2.17	“	0.98	0.88	1	0.98	885
25	B&D-M(P11′-P3″1′	4693	7.18	2.17	“	0.92	0.88	3	0.98	1966

Table 5.7: Average passenger car speed, Density and Level of service of each line

No	Assigned road segment	VP	FFS(km/h)	S	D=Vp/s	LOS
1	K-M (P1)	591	60	60	9.9	B
2	K-B (p2-P4′)	1084	75	75	14.5	C
3	K-B (P4′-P5′)	872	70	70	12.5	C
4	K-D&M(p2)	1452	55	55	26.4	E
5	K-M(p3)	2104	60	60	35.1	F
6	K-D (p3-P3′)	798	70	70	11.4	B
7	M-B (p4)	447	42	42	10.6	B
8	M-D (p4)	1268	70	70	18.1	D
9	D-M (p5-P8)	2519	68	68	37.0	F
10	D-B (p5-P5′)	1249	60	60	20.8	D
11	D-M&B (p6-P5)	2096	70	70	29.9	F
12	D-M (p7)	489	40	40	12.2	C
13	D-B (p7)	638	50	50	12.8	C
14	D-K (p8-P8′)	1342	42	42	32.0	F
15	D-M (p8-P11′)	1848	70	70	26.4	E
16	B-K (p9-P8′)	2106	70	70	30.1	E
17	B-K (P8′-P12)	1851	70	70	26.4	E
18	B-D&M (p9-P11)	1530	50	50	30.6	F
19	B-D (P10)	259	50	50	5.2	A
20	B-M (P11-P11′)	1101	45	45	24.5	E
21	B-D (P11-P3′)	428	60	60	7.1	B
22	M-D (P12-P4)	1492	70	70	21.3	D
23	M-K (P12-P12′)	1007	45	45	22.4	D
24	M-K (P13-P13′)	885	45	45	19.7	D
25	B&D-M(P11′-P3″1′	1966	70	70	28.1	E

Table 5.8: Capacity of Merge influence area

No.	Junction	VF	N	LA	PFM	$V12 = VF * PFM$	VR	$VR12 = V12 + VR$
1	Exit of p4’	2168	2	190	1	2168	447	2615
2	Exit of p8’	4212	3	190	0.595	2506	1342	3848
3	Exit of p5’	2616	3	190	0.595	1557	1249	2806
4	Exit of p3’andp11’	2536	2	190	1	2536	1654	4190
5	Exit of p3’’and p1’’	5898	3	190	0.595	3509	2695	6204
6	Exit of p12’andp13’	5553	3	190	0.595	3304	2899	6203
7	Exit of p11’	3696	3	190	0.595	2199	2202	4401

Table 5.9 LOS of merge influence area

No.	Junction	LA	$V12$	VR	$D_R = 3.402 + 0.00456VR + 0.0048V12 - 0.01278LA$	LOS
1	Exit of p4’	190	2168	447	13.4	C
2	Exit of p8’	190	2506	1342	19.1	D
3	Exit of p5’	190	1557	1249	14.1	C
4	Exit of p3’andp11’	190	2536	1654	20.7	D
5	Exit of p3’’and p1’’	190	3509	2695	28.7	E
6	Exit of p12’andp13’	190	3304	2899	29.1	E
7	Exit of p11’	190	2199	2202	21.6	D

Table 5.7 shows that flow rate and LOS of each line of the interchange. LOS of the lines will be between within A-F grade. One line will have A, four lines will have B, four lines will have C, five lines will have D, six lines will have E and five lines will have F.From table 5.9 exit of Meskel Square and Kera will have level of service of E.

6.0. Conclusion and recommendation

6.1. Conclusion

Capacity analysis results of Gotera interchange indicates

- Currently all ramps and main lines are under saturated that means as shown data analysis results from 25 lines of the interchange 12 lines have LOS A 7 lines have LOS B and the rest 6 lines have LOS of C.
- Exit of Meskel Square and kera have a LOS of C Shows stable flow not free flow than the others.
- In the future after 10 years, One line will have A, four lines will have B, four lines will have C, five lines will have D, six lines will have E and five lines will have F. most of the interchange lines will be unstable and volatile flow condition.
- Exit of Meskel Square and Kera will have LOS of E.
- Currently, the overall condition of the interchange is safe and to the future after 10 years, will not give a satisfactory service.
- From traffic data most of traffic categories are taxi and private cars.

6.2. Recommendations

- To the future most of the interchange lines will not give a satisfactory service, so to control this overstated traffic flow and to continue the current condition of traffic flow and also decrease traffic congestion, people shall use rail way transportation.
- Currently , railway lines are constructed from piassa to Nifas silk that means this lines through Meskel-Sqaure to Deber-Zeyt,this will share (reduce) number of taxi and private cars. In other words their mode of transportation must be changed from taxi and private cars to rail way transport so, in this direction it will be no change
- In the direction of Bole to Kera there is no railway lines , additional lines will be required at Exit of p12’andp13’ towards Kera

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Acronyms and Abbreviations

Notation	Description
fLW	Adjustment for lane width, km/h
fLC	Adjustment for lateral clearance km/h
fN	Adjustment for number of lanes, km/h
fID	Adjustment for interchange density, km/h
fHV	Heavy vehicle adjustment factor
fp	Adjustment factor for presence of occasional or non-familiar users of a facility
PT	Portion of trucks and buses
PR	Portion of recreational vehicles
ET	Passenger car equivalency for trucks
ER	Passenger car equivalency for recreational vehicle
S	Average speed
DR	Density of merges and diverge influence area
L_{UP}	Distance to adjacent upstream ramp
L_{down}	Distance to adjacent downstream ramp
P_{FM}	Proportion of approaching freeway flow remaining in lane 1 and 2 immediately Upstream of merge
P_{FD}	Proportion of through freeway remaining in Lanes 1 and 2 immediately upstreamof diverge
V_P	Demand flow rate under equivalent ideal conditions, pc/h/ln
V_F	Maximum total flow approaching a merge or diverge area on the freeway
V_{FO}	Maximum total flow departing from a merge or diverge area on the freeway
V_{R12}	Maximum total flow entering the ramp influence area for merge areas and
V_R	Maximum flow on a ramp

Appendix A: Abbreviations

AACRA	Addis Ababa city road Authority
AADT	Average annual daily traffic
AASHTO	American Association of State Highway and Transportation Officials
ADT	Average Daily Traffic
BFFS	Base free-flow speed of the freeway
DDHV	Direction Design Hourly Volumes
DHV	Design Hour Volume
EAC	East Africa Community
EATTFP	East African Trade and Transport Facilitation Project
ERC	Ethiopia railways Corporation
FDOT	Florida Department of Transportation
FFS	Free flow speed
HCM	Highway capacity manual
L_A	Length of acceleration lane
LD	Length of deceleration lane (m)
MSFi	Maximum service flow rate for level-of-service 'I"
N	Number of lanes (in one direction) on the facility
PCE	Passenger car equivalency
PHF	Peak –hour factor
SFi	Service flow rate for level of service "I"
SFR	Free-Flow Speed of Ramp,

Appendix B: Definition

K30 – is proportion of AADT occurring during the 30^{th} highest hour of the design year.

D30 – is proportion of traffic in the 30th highest hour of the design year traveling in the peak direction.

PHF- is the hourly volume during the analysis hour divided by the peak 15-minute flow rate Within the analysis hour.